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MATERIALS TESTING

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DEPARTMENTS OF THE ARMY, THE NAVY, AND THE AIR FORCE

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NO. 89-3

DEPARTMENTS OF THE ARMY,
THE NAVY, AND THE AIR FORCE

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CHAPTER 1

INTRODUCTION

1–1. Purpose and Scope

a. The purpose of this manual is to provide technical information to enable military personnel to take samples and perform engineering tests and calculations on soils, bituminous paving mixtures, portland cement concrete, stabilized soils, and their component aggregates and cements. These tests and calculations are required to achieve proper design with these materials and adequate control over their use in military construction.

b. The materials covered in this annual include soils, aggregates, bituminous cements, portland cement, bituminous paving mixtures, portland cement concrete, and stabilized soil including stabilizing agents such as bitumens, cements, lime, flyash, and chemical modifiers. Detailed instructions are given for taking adequate representative test samples. Step-by-step procedures are given for making physical properties tests and recording, calculating and evaluating the test results. Methods for design of bituminous paving mixtures, portland cement concrete and soil stabilization are explained, together with the procedures and tests required for control of the manufacture of these mixtures. The tools and equipment for performing these tests are described. General instructions relative to the care, calibration, and use of test equipment are contained in the manual.

c. Users of this publication are encouraged to submit recommended changes and comments to improve the publication. Comments should be keyed to the specific page, paragraph, and line of the text in which the change is recommended. Reasons will be provided for each comment to insure understanding and complete evaluation. Comments should be prepared using DA Form 2028 (Recommended Changes to Publications) and forwarded direct to the Commandant, U.S. Army Engineer School, Fort Belvoir, Va. 22060.

d. This manual is adopted for use by Navy personnel as outlined in the following subparagraphs:

(1) Certain tests and procedures prescribed in this manual differ in principle or method and/or are more detailed than counterpart tests currently required by the Navy for new construction at Navy installations including those in forward areas.

(2) Although this manual provides general guidance for materials and soils testing, the design of new structures and facilities will be based on the results obtained from methods and procedures outlined in NAVDOCKS DM–7 "Soil Mechanics, Foundations and Earth Structures." Additional criteria related to exploration and sampling are found in NAVFAC DM–5 under "Exploration for Highways" and DM–21 under "Exploration for Airfields." When methods and procedures prescribed by the Navy differ from those in this manual, the Navy methods will take precedence.

1–2. Discussion of Materials Testing

a. The military engineer is responsible for developing and maintaining the means of transportation and access, housing and special structures, sanitary facilities, and military defenses. Transportation facilities include roads, railways, airports and landing strips, pipelines, and harbor structures. These structures and facilities all are built on, or in some cases are made of the local soil. To design these, the engineer must know the type of soil at the site and its characteristic properties. For example, in designing a road or airfield, he must determine whether the soil can withstand the loads, including the vehicle weights to be transported, or whether soil stabilization or paving will be needed. If the road or airfield requires a pavement, then the suitability of available aggregate materials must be established, since speed and efficiency of construction dictate the use of nearby sources. Bituminous paving mixtures and portland cement concrete are made at or near the work site, under the control of the local engineering officers who must design the mix and test the performance of the finished product. The engineer must have basic data concerning the properties of these materials in order to use them effectively in construction. Such data are obtained from the tests described in this manual.

b. The properties of all materials are determined by their chemical composition and the physical structure in which the constituent compounds are arranged. Earth minerals and cementing materials are very complex and the nature of the forces that bind them together is but imperfectly understood on an atomic or molecular scale. However, the strength, stiffness, stability, and resistance to wear, erosion, or weathering can be determined by material tests on the bulk material. Laboratory research related to field observation, and experience with such materials in service, enable engineers to establish limiting values of the properties measured in these tests that will insure satisfactory performance of the material in service. Materials specifications based on this research give such limits. Tests of representative samples of a particular material available for an engineering use then are made and the results are compared to the specifications to decide whether the material will do for the intended application. Materials tests also are used to identify or classify materials on the basis of their physical properties. These tests also provide basic data on the aggregates and cements necessary for the design of bituminous mixtures, stabilized soil, or portland cement concrete.

c. The physical characteristics of soil that determine its usefulness to support traffic or to serve as a subgrade or foundation material depend upon the size and shape of the individual grains, the grain size distribution, the specific gravity, and compaction characterstics. Properties of many soils are strongly dependent upon moisture content and tests for the moisture limits describe their plasticity characteristics. Strength tests such as California bearing ratio and unconfined compression measure load-carrying capacity directly. Tests for these properties include expedient and deliberate testing procedures and are used to identify and classify the type of soil represented by the samples. With the soil accurately identified and described by the results of these tests, its suitability for supporting traffic, as a subgrade, base, or foundation material, or as an aggregate, filler, or binder for mixtures, can be evaluated. The construction of subgrades and bases for pavements, embankments, and other earth structures requires continual testing during the course of the work to insure adequate compaction and to adjust mixtures and construction methods as needed.

d. Tests and evaluation of results are more easily made by using some common reference or system which has a universal interpretation. Thus no matter who made the tests or the evaluation, the results would be understandable by anyone familiar with the system. One reference for soils used by the military engineer is called the *Unified Soil Classification System*. All soils are divided into three major categories, two of which are based on grain size. Further subdivision distinguishes between gravel, sand, silt clay, and organic content and between well-graded and poorly-graded soils. Each of these types is symbolized by a combination of two or four letters. A more detailed explanation of the Unified Soil Classification System will be found in section I of chapter 2 and in TM 5–330.*

e. The engineer, having the soil classification complete and with all the other information required for a proper design of an airfield or road, can start the design. The design requirements are covered in TM–5–330. Design requirements and criteria for Navy airfields are covered in NAVFAC DM–21, "Design Manual—Airfield Pavements." Evaluation and strengthening of existing airfield pavements is covered in chapter 4 of NAVFAC DM–21.

f. Bituminous paving mixtures consist of aggregates, filler, and bitumen binder. Aggregate sources near the construction area are tested to give data on particle size distribution and specific gravity; these data are used in designing the mix. Testing of bituminous cements as received at the mixing plant may be required to identify the material, to determine its suitability as a binder, or to provide data to determine what aggregates and fillers are required. The tests of bitumens described in this manual are field identification procedures to expedite use of the material until more detailed tests can be performed.

g. Portland cement concrete is a mixture of fine and coarse aggregates, portland cement, and water. The cement and water react chemically to form compounds that hold the aggregates in a strong rock-like mass. Concrete is made in mixing plants, field mixers, or truck mixers near or at the construction site. The quality of the concrete produced depends upon proper mix design to achieve the desired workability of the fresh concrete and strength of the hardened material. Tests of aggregates and fresh and hardened concrete, and instructions for using these test data in mix design and control, are given in this manual.

h. Accurate soil description given by tests is necessary to determine whether its properties must be improved by "stabilization" to make it adequate for supporting traffic or use as a base course. Test results indicate the method of stabilization and materials to be used, as well as verifying the adequacy of the stabilized soil.

i. The test procedures and terminology used in this manual conform to the latest methods and specifications of the American Society for Testing and Materials, the American Association of State Highway Officials, the American Concrete Institute, and the Portland Cement Association, where appropriate.

j. The tests given in this manual also apply to arctic construction. However, cold weather effects present different problems and additional tests will be required for correct evaluation of the materials. These additional tests as well as the considerations associated with arctic construction will be found in TM 5–349.

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*The references in this manual are listed in Appendix A.

1-3. Equipment

The equipment for materials testing consists of three sets: a soil test set, an asphalt test set, and a concrete test set. The soil test set is considered the basic set and the other two sets are used in conjunction with it. In selected operations, for example control testing at concrete batch plant, the concrete test set can be used independently of the soil test set. Certain items listed as part of the sets are not issued with the sets but must be requisitioned separately. Some of the test methods, such as the specific gravity of solids, the hydrometer analysis, and the shrinkage limit, describe items of equipment which are not issued with the test sets but would be either locally available or procurable from commercial sources.

a. Test Set, Soil (SC 6635-97-CL-E02)*. This is the basic set for performing soil tests, including sieve analysis, moisture content, Atterberg limits, California bearing ratio (CBR), and soil trafficability. Included in this test set is the separately packaged test set, soil trafficability (SC 6635-97-CL-E01)*. Items such as DA forms, pencils, tracing paper, labels and tags, towels, twine, and wax are not issued initially with the set, but must be requisitioned separately by stock or form number.

b. Test Set, Asphalt (SC 6635-97-CL-E03). The asphalt set is issued in three chests: a laboratory centrifuge chest, a miscellaneous equipment chest, and a 1,1,1-Trichloroethane chest. This set is not complete within itself and therefore must be used in conjunction with the soil test set (*a* above). Items such as forms, brushes, cloth, and chemicals (alcohol, sodium hydroxide, and 1,1,1-Trichloroethane) are not issued with the set but must be requisitioned separately.

c. Test Set, Concrete (SC 6635-97-CL-E04). The concrete set is issued in three chests: a beam testing-machine chest, a collapsible steel forms chest, and a miscellaneous equipment chest. This set normally is used in conjunction with the soil test set (*a* above). DA forms, chemicals, and some other items are not issued with the set but must be requisitioned separately.

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*These sets are no longer standard items of issue; new items are being developed.

CHAPTER 2

SOILS

Section I. CLASSIFICATION, EXPLORATION, AND FIELD IDENTIFICATION

2-1. Description

The soil types in an area are a very important consideration in selecting the exact location of any structure. The military engineer, construction supervisor, and members of engineer reconnaissance parties must be capable of properly identifying soils in the field to determine their engineering characteristics. Because the military engineer must be economical of time, men, equipment, and material, as well as money, site selection for a project must be made with these factors in mind.

a. Soil, by definition, is a heterogeneous accumulation of uncemented or loosely cemented mineral grains inclosing voids of varying sizes. These voids may contain air, water, organic matter, or different combinations of these materials in varying amounts. The engineer must therefore concern himself not only with the sizes of the particles, but also with the voids between them and particularly what these voids inclose (water, air, or organic materials).

b. Geologists classify rocks into three basic groups: igneous, formed by cooling from a molten state; sedimentary, formed by accumulation and cementation of existing particles and remains of plants and animals; and metamorphic, formed from existing rocks subjected to heat and pressure. Exposed to the atmosphere, these rocks disintegrate and decompose either by mechanical action (wind, water, ice, and vegetation) or chemical action, or both in combination. The resulting material may remain where it is formed or it may be transported by water, glaciers, wind, or gravity and deposited at a distance from the parent rock (TM 5-545). This resulting material is called soil.

c. The process of soil formation is a continuing one and is still in action today. The great number of original rocks, the variety of soil-forming forces, and the time that these forces have acted produce a tremendous number of different soils. For engineering purposes, the soils are evaluated by their basic physical properties: gradation or sizes of the different particles, bearing capacity as reflected by soil density, and particle shapes. The engineer extends his evaluation by considering the effect of water action on the soils. With a complete evaluation, the engineer can determine whether the soil is adequate for his project.

d. At a particular location there usually will be several layers or strata, one above the other, each of which is composed of a different kind of soil. Strata may be a few inches or many feet thick. The topmost layer is called the agricultural soil since it supports plant growth. Adequate soil evaluation for engineering uses requires that all strata be identified to whatever depth may be affected by the construction. A vertical cross section through the earth, with the depths and types of soil indicated, is called a soil profile. See TM 5-330, chapter 2, section IV for more detailed description.

2-2. Physical Properties of Soil

The physical properties of a soil aid in determining the engineering characteristics. These properties are the basis for the system of soil classification used in engineering identification of soil types. Physical characteristics of soil particles are size and shape. The proportions of particles of different sizes determine the gradation of the aggregate of soil particles. Compactness refers to the closeness of packing of the soil particles; the closer the packing, the greater the compactness and the larger the weight of soil per unit of volume. Plasticity characteristics of fine-grained soil components include liquid and plastic limits, shrinkage ratios, dry strength, and unconfined compressive strength. Specific gravity of soil particles aids in their identification. The presence of organic matter is important to the engineering use of soil materials. Color, texture, odor, structure, and consistency are readily observed factors that aid in soil description.

a. Grain Size Groups. Soils are divided into groups based on the size of the particle grains in the soil mass. Common practice distinguishes the sizes through the use of sieves. A sieve is a screen attached across the end of a shallow cylindrical frame. The screen permits particles smaller than the openings to fall through and retains the larger particles on the sieve. By using sieves with screen openings of different sizes (the largest on the top and the smallest at the bottom), the soil can be separated into particle groups based on size. The amount remaining on each sieve is measured and described as a percentage by weight of the entire sample. Size groups as employed in the United Soil Classification System are as follows:

Size groups	Sieve size	
	Passing	Retained on
Cobbles	No maximum size*	3-inch
Gravels	3-inch	No. 4
Sands	No. 4	No. 200
Fines	No. 200	No minimum size

*In military engineering, maximum size of cobbles is accepted as 40 inches, based upon maximum jaw opening of the crushing unit.

Particles passing the No. 200 sieve but larger than 0.002-mm to 0.005-mm are called silt and those finer are clays.

b. Particle Shape. The shape of the particles influences the strength and stability of a soil. Two general shapes are normally recognized—bulky and platy (fig. 2–1). Bulky shape includes particles which are relatively equal in all three dimensions as compared to the platy shape in which one dimension is very small compared to the other two. *For example*, a thick book would be considered bulky whereas a page of the book would be platy.

(1) Bulky shapes are subdivided depending on the amount of weathering which has acted on them. They may be angular, subangular, subrounded, or rounded (fig. 2–1). The angular shape shows flat surfaces, jagged projections, and sharp ridges, whereas the rounded shape has smooth curved surfaces and almost approaches a sphere. Cobbles, gravel, sand, and silt fall into the bulky shape group.

(2) Platy grain shapes have one dimension relatively small compared to the other two. The faces at approximately right angles to this dimension will have relatively large areas whereas the other surface will be edges and present small surfaces. Clay exhibits this shape. Coarse-grained soil particles with bulky shapes are individually discernible to the naked eye, whereas fine grained particles with platy or bulky shapes are not.

c. Soil Gradation. The size and shape of the soil particles discussed above deal with properties of the individual grains in a soil mass. Gradation describes the distribution of the different size groups within a soil sample. The soil may be well graded or poorly graded.

(1) A soil, in order to be classified as well graded, must have a good range of all representative particle sizes between the largest and the smallest. All sizes are represented and no one size is either overabundant or missing (fig. 2–2).

(2) Poorly graded soils are either those containing a narrow range of particle sizes or those with some intermediate sizes lacking (fig. 2–2). Soils with a limited range of particle sizes are called "uniformly graded." Soils which have some intermediate size or sizes not well represented or missing are called "gap graded", "step graded", or "skip graded".

d. Compactness. The structure of the aggregate of soil particles may be dense, in which the particles are closely packed and may be said to have a high degree of compactness; or the structure of the aggregate of soil particles may be loose, in which case the particles are not fitted together as closely as possible and, hence, lack compactness. A dense structure provides "interlocking" of particles with smaller grains filling the voids, or open spaces, between the larger particles. When each particle is closely surrounded by other particles the grain-to-grain contacts are increased, the tendency for displacement of individual grains under load is lessened, and the soil is capable of supporting heavier loads. Coarse materials that are well graded usually are dense and have strength and stability under load. Loose, open structures have large void spaces and will compact under load, leading to settlement or disintegration under foundation or traffic loads.

e. Specific Gravity of Soil. The *specific gravity*, designated by the symbol G_s, is defined as the ratio between the weight per unit volume of the material and the weight per unit volume of water at a stated temperature, usually 20° C. Using the metric system, specific gravity can be determined by using the following formula:

$$\text{Specific gravity (metric system)} = \frac{\text{weight of sample in air (gm)}}{\text{weight of sample in air (gm)} - \text{weight of sample submerged (gm)}}$$

Test procedures are given in section IV of this chapter. The specific gravity of the solid substance of most inorganic soils varies between 2.60 and 2.80. Tropical iron-rich laterite as well as some lateritic soils will generally have a specific gravity of 3.0 or

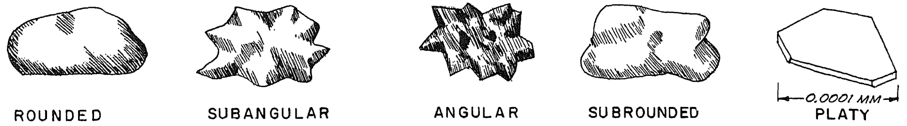


Figure 2–1. Shapes of soil grains.

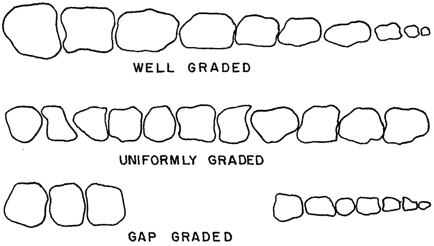


Figure 2–2. Types of soil gradations.

more. Sand particles composed of quartz have a specific gravity around 2.65. Clays can have values as high as 3.50. The solids of soil particles are composed of minerals. Generally these minerals will have a specific gravity greater than 2.60. Therefore, values of specific gravity smaller than that are an indication of the possible presence of organic matter.

f. Soil Moisture. The moisture content of a soil mass is often the most important factor affecting the engineering characteristics of the soil. The water may enter from the surface or may move through the subsurface layers either by gravitational pull, by capillary action, or by hygroscopic action. This moisture, present in most cases, influence various soils differently, and may have the greatest implication upon the soil's behavior when subjected to loading.

(1) Surface water results from precipitation or runoff and enters the soil through the openings between the particles. This moisture may adhere to the different particles or it may penetrate the soil to some lower layer.

(2) Subsurface water is collected or held in pools or layers beneath the surface by a restricting layer of soil or rock. This water is constantly acted upon by one or more external forces.

(*a*) Water controlled by gravity (*free* or *gravitational water*) seeks a lower layer and moves through the voids or spaces until it reaches some restriction. This restriction may be a bedrock or an impervious layer of soil whose openings or voids are so small as to prevent water passage.

(*b*) The voids or spaces in a soil may form continuous tunnels or tubes and cause the water to rise in the tubes by capillary action (*capillary moisture*). Since the smaller the tube, the stronger the capillary action, the water will rise higher in the finer soils which have smaller interconnected voids. This area of moisture above the free water layer or pool is called the *capillary fringe.*

(*c*) Another force acting on soil water is absorption by the atmosphere. As the moisture evaporates from the soil's surface, it draws more moisture from the soil below. This process will continue until the soil moisture reaches an equilibrium with the water vapor in the air. The amount of water held by the soil is called the *hygroscopic moisture.*

(3) To define the amount of water present in a soil sample, the term *moisture content* (symbol w) is used. It is the proportion of the weight of water to the weight of the solid mineral grains (weight of dry soil) expressed as a percentage or

$$w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100\%$$

(4) The effects of soil moisture to a great extent depend upon grain size. Coarse-grained soils with the larger voids permit easy drainage of water. They are less susceptible to capillary action. The amount of water held in these soils is less than in fine-grained soils since the surface area is smaller and excess water will tend to drain off whenever possible. The fine grains and their small voids retard the movement of water and also tend to hold the water by surface tension. Many fine-grained soils made from certain minerals exhibit plasticity (putty-like properties) within a range of moisture contents. These soils are called clays and their properties may vary from essentially liquid to almost brick-hard with different amounts of moisture. Further, clays are essentially impervious to the passage of free or capillary moisture.

g. Cohesive Soils. A cohesive soil has considerable strength when air-dried, but has low strength when its moisture content is high. Such clay soils are composed of fine-grained (usually less than 0.002-mm) particles of so-called clay minerals. Clay particles usually have flat, platy shapes, and are capable of holding a film of absorbed water on their surfaces. Absorbed water is held by physiochemical forces and has properties substantially different from ordinary or chemically combined water. The attraction exerted by clay particles for water molecules gives these materials the property of plasticity. Plasticity is a property of the fine-grained portion of a soil that allows it to be deformed beyond the point of recovery without cracking or appreciable volume change. This property permits a clay to be rolled into thin threads at some moisture content without crumbling. Note that only clay minerals possess this property, while some minerals, such as quartz powder, cannot be made plastic no matter how fine the particles nor how much water is added. Thus, the degree of plasticity is a general index to the clay content of a soil. The terms "fat" and "lean" are sometimes used to distinguish between highly plastic and slightly plastic soils.

For example, lean clay is only slightly plastic whereas fat clay is highly plastic.

(1) In engineering practice, soil plasticity is determined by observing the different physical states that a plastic soil passes through as the moisture

conditions change. The boundaries between the different states as described by the moisture content at the time of change are called *consistency limits* or *Atterberg limits*, named after the Swedish scientist who defined them years ago.

(2) The *liquid limit* (LL) is the moisture content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil. Above this value, the soil is presumed to be a liquid and behaves as such by flowing freely under its own weight. Below this value, it will deform under pressure without crumbling, provided the soil exhibits a plastic state.

(3) The *plastic limit* (PL) is the moisture content at an arbitrary limit between the plastic and semisolid states. As the sample is dried, the semisolid state is reached when the soil is no longer pliable and crumbles under pressure. Between the liquid and plastic limits is the *plastic range*, and the numerical difference in moisture contents between the two limits is called the *plasticity index* (PI) (PI=LL−PL). It defines the range of moisture content in which the soil is in a plastic state.

(4) The *shrinkage limit* is the boundary in moisture content between the solid and the semisolid states. The solid state is reached when the soil sample, upon being dried, finally reaches a limiting or minimum volume. Beyond this point, further drying will not reduce the volume, but may cause cracking. The limit tests are described in section IX of this chaper.

h. Organic Soils. Soils having a high content of organic material are described as organic soils. They usually are very compressible and have poor load-maintaining properties.

i. Effects of Soil Characteristics. The soil characteristics are a measure of the soil's suitability to serve some intended purpose. Generally, a dense soil will withstand greater applied loads (have greater bearing capacity) than a loose soil. Particle size has a definite relation to this capacity. From empirical tests, it has been found that well graded, coarse grained soils generally can be compacted to a greater density than fine grained soils, because the smaller particles tend to fill the spaces between the larger ones. The shape of the grains also affects the bearing capacity. Angular particles tend to interlock and form a denser mass and are more stable than the rounded particles which can roll or slide past one another. Poorly graded soils with their lack of one or more sizes leave more or greater voids and therefore a less dense mass. Moisture content and the consistency limits aid in describing the soil's suitability. A coarse grained sandy or gravelly soil generally has good drainage characteristics and may be used in its natural state. A fine grained clayey soil with a high plasticity index may require considerable treatment especially if used in a moist location.

2–3. Classification

Soils seldom exist in nature separately as sand, gravel, or any other single component. They are usually mixtures with varying proportions of different size particles. Each component contributes to the characteristics of the mixture. The *Unified Soil Classification System* is based on the characteristics which indicate how a soil will behave as a construction material.

a. In the system, all soils are divided into three major categories: coarse grained, fine grained, and peat. The first two are differentiated by grain size, whereas the third is identified by the presence of large amounts of organic material.

b. Each of the major categories is further subdivided into groups and a letter symbol is assigned to each group.

For example, the symbol assigned to gravel is G, to sand—S, to silt—M, and to clay—C. Refinements in the system also distingush between well graded (W) and poorly graded (P) soils, and between high-compressibility (H) and low-compressibility (L). Highly organic soils, such as peat, are symbolized Pt, whereas organic silts and clays use the letter O as a prefix. These symbols are combined to describe soil mixture.

c. The physical properties are determined by appropriate tests and calculations; these characteristics are used to classify the soil. The criteria for identifying the different soil types are give in table B–1 (App. B), which has been especially developed to facilitate soil classification, and in the following paragraphs. The chart of figure B–1 (App. B) shows the procedures to be followed. Principal categories shown in the chart include soil groups, group symbols, and typical soil names; laboratory classification criteria; field identification procedures; and information required for describing soils. These charts provide a simple, systematic means of soil classification. Once an accurate classification is attained, the potential behavior of the soil under traffic or foundation loads is known.

2–4. Coarse Grained Soils

a. Definition. Coarse grained soils are defined as those in which at least half the material by weight is larger than (retained on) a No. 200 sieve. They are divided into two major divisions: gravels and sands. A coarse grained soil is classed as a gravel if more than half the coarse fraction by weight is retained on a No. 4 sieve. It is a sand if more than half the coarse fraction is smaller than a No. 4 sieve. In general practice there is no clear-cut boundary between gravelly and sandy soils; as far as behavior is concerned, the exact point of division is relatively unimportant. Where a mixture occurs, the primary name is the predominant fraction (in percent by weight) and the minor fraction is used as an adjective.

For example, a sandy gravel would be a mixture containing more gravel than sand by weight. For the purpose of systematizing the discussion, it is desirable to further divide coarse-grained soils into three groups on the basis of the amount of fines (materials passing a No. 200 sieve) which they contain.

NOTE

> If fines interfere with free draining properties, as may occur with plastic fines, use double symbol (that is, GW–GC, and so on) meaning that such soils will be classed with soils having from 5 to 12 percent nonplastic fines or 0 to 12 percent plastic fines.

b. Coarse-Grained Soils With Less Than 5 Percent Nonplastic Fines. These soils may fall into the groups GW, GP, SW, or SP, as follows, where the shape of the grain-size distribution curve determines the second letter of the symbol. Paragraph 2–2 discusses the terms well graded (W) and poorly graded (P). However, as denoted above, if the fines do interfere with the free drainage properties, a dual or double symbol will be used.

(1) *GW and SW groups.* In the GW groups are well graded gravels and gravel-sand mixtures which contain little or no nonplastic fines. The presence of the fines must not noticeably change the strength characteristics of the coarse-grained fraction, and must not interfere with its free-draining characteristics. The SW groups contain well graded sands and gravelly sands with little or no plastic fines.

(2) *GP and SP groups.* The GP group includes poorly graded gravels and gravel-sand mixtures containing little or no nonplastic fines. In the SP group are contained poorly graded sands and gravelly sands with little or no nonplastic fines. These soils will not meet the gradation requirements established for the GW and SW groups.

c. Coarse-Grained Soils Containing More Than 12 Percent Fines. These soils may fall into the groups designated GM, GC, SM, and SC. The use of the symbols M and C is based upon the plasticity characteristics of the material passing the No. 40 sieve. The liquid limit and plasticity index are used in specifying the laboratory criteria for these groups. Reference also is made to the plasticity chart shown in table B–1 which is based upon established relationships between the liquid limit and plasticity index for many different fine-grained soils. The symbol M is used to indicate that the material passing the No. 40 sieve is silty in character. M usually designates a fine-grained soil of little or no plasticity. The symbol C is used to indicate that the binder soil is predominantly clayey in character.

(1) *GM and SM groups.* Typical of the soils included in the GM group are silty-gravels and gravel-sand-silt mixtures. Similarly, in the SM group are contained silty sands and sand-silt mixtures. For both of these groups, the Atterberg limits must plot below the A-line of the plasticity chart (table B–1) or the plasticity index (para 2–61) must be less than 4.

(2) *GC and SC groups.* The GC group includes clayey gravels and gravel-sand-clay mixtures. Similarly, SC includes clayey sands and sand-clay mixtures. For both of these groups, the Atterberg limits must plot above the A-line with a plasticity index of more than 7. See section IX of this chapter.

d. Borderline Soils. Coarse-grained soils which contain between 5 and 12 percent of material passing the No. 200 sieve are classed as borderline and are given a dual symbol (for example, GW–GM). Similarly, coarse-grained soils which contain more than 12 percent of material passing the No. 200 sieve, and for which the limits plot in the shaded portion of the plasticity chart (table B–1), are classed as borderline and require dual symbols (for example, SM–SC). It is possible, in rare instances, for a soil to fall into more than one borderline zone and, if appropriate symbols were used for each possible classification, the result would be a multiple designation consisting of three or more symbols. This approach is unnecessarily complicated. It is considered best to use only a double symbol in these cases, selecting the two that are believed to be most representative of the probable behavior of the soil. In cases of doubt, the symbols representing the poorer of the possible groupings should be used. For example, a well-graded sandy soil with 8 percent passing the No. 200 sieve, with LL 28 and a PI of 9, would be designated as SW–SC. If the Atterberg limits of this soil were such as to plot in the shaded portion of the plasticity chart (*for example*, LL 20 and PI 5), the soil would be designated either SW–SC or SW–SM, depending on the judgment of the engineer, from the standpoint of the climatic region he is in.

2–5. Fine-Grained Soils

a. Definition. Fine-grained soils are those in which more than half the material is smaller than (passes) a No. 200 sieve.

b. Groupings. The fine-grained soils are not classified on the basis of grain size distribution, but according to plasticity and compressibility. Laboratory classification criteria are based on the relationship between the liquid limit and plasticity index which is designated as the plasticity chart in table B–1. This chart was established by the determination of limits for many soils, together with an analysis of the effect of limits upon physical characteristics. Examination of the chart will show that there are two major groupings of fine-grained soils. These are the L groups, which have liquid limits less than 50, and the H groups, which have liquid limits equal to and greater than 50. The symbols L and H have general meanings of low and high compressibility, respectively. Fine-grained soils are further divided with relation to their position above or below the A-line of the plasticity chart.

(1) *ML and MH groups.* Typical soils of the ML and MH groups are inorganic silts; those of low compressibility in the ML group, others in the MH. All of these soils plot below the A-line. In the ML group are included very fine sands, rock flours (rock dust), and silty or clayey fine sands or clayey silts with slight plasticity. Loess-type soils usually fall into this group. Micaceous and diatomaceous soils generally fall into the MH group, but may extend into the ML group when their liquid limits are less than 50. The same statement is true of certain types of kaolin clays which have low plasticity. Plastic silts will fall into the MH group.

(2) *CL and CH groups.* In these groups, the symbol C stands for clay, with L and H denoting low or high liquid limits. These soils plot above the A-line and are principally inorganic clays. In the CL group are included gravelly clays, sandy clays, silty clays, and lean clays. In the CH group are inorganic clays of high plasticity, including fat clays, the gumbo clays of the southern United States, volcanic clays, and bentonite. The glacial clays of the northern United States cover a wide band in the CL and CH groups.

(3) *OL and OH groups.* The soils in these two groups are characterized by the presence of organic matter, hence the symbol, O. All of these soils generally plot below the A-line. Organic silts and organic silt-clays of low plasticity fall into the OL group, while organic clays of high plasticity plot in the OH zone of the plasticity chart. Many of the organic silts, silt-clays, and clays deposited by the rivers along the lower reaches of the Atlantic seaboard have liquid limits above 40, and plot below the A-line. Peaty soils may have liquid limits of several hundred percent, and will plot well below the A-line due to their high percentage of decomposed vegetational matter. A liquid limit test, however, is not a true indicator where a considerable portion consists of other than soil matter.

(4) *Borderline soils.* Fine-grained soils which have limits which plot in the shaded portion of the plasticity chart are borderline cases, and are given dual symbols (*for example*, CL–ML). Several soil types, exhibiting low plasticity, plot in this general region on the chart, where no definite boundary between silty and clayey soils exists.

2–6. Highly Organic Soils

A special classification (Pt) is reserved for t
highly organic soils, such as peat, which have
many undesirable characteristics from the standpo
of their behavior as foundations and their use as cc
struction materials. No laboratory criteria are esta
lished for these soils, as they generally can be read
identified in the field by their distinctive color a
odor, spongy feel, and frequently, fibrous textu
Particles of leaves, grass, branches, or other fibr
vegetable matter are common components of th
soils.

2–7. Soil Exploration

Exploration and sampling are basic operations
engineering work involving soils. The samples m
be required to determine the type of structure wh
can be built on the soil or to control the construct
material. Soil exploration programs which determ
which samples are to be taken are discussed in det
in TM 5–330. The methods of taking samples
described in paragraphs 2–11 through 2–22.

a. When circumstances and time permit, the s
exploration program develops a "picture" of
soil conditions at the site. Available sources of
formation such as agricultural soil maps and repo
geologic maps and reports, topographic maps, ae
photographs, or intelligence reports are examin
Trained experts often can identify soil types fr
aerial photographs combined with other informat
from the above sources. Soil exploration on
ground is necessary, however, to provide quantitat
test data on the available materials.

b. Soil exploration involves field reconnaissa
field identification and classification, and samplin
time and circumstances permit. Soil surveys for
foundations of structures are relatively limited
area but require knowledge of the soil strata t
depth that depends upon the nature of the struct
Soils under earth structures, or embankments, in
cations where deep cuts will be made and in depo
that are potential sources of aggregates, must be
plored to greater depths. Programs for these surv
must be specified by the foundation engineer,
often must be developed as partial data are obtai
Subgrade soil surveys for highways and airp
should include location and depth of all soil ty
along the route with complete descriptions and cla
fications. The moisture content and in-place den
of the soils should be determined. Properly
formed soil exploration provides the engineer w
information to decide on the final location of
road, suitable fill material, the design of the road
cross section, drainage requirements, subgrade st
lization if needed, and the type of road surface.

c. The exploration is conducted to determine
following: location, nature, and classification of

layers; condition of soils in place; drainage characteristics; and ground water and bedrock. For a complete picture, exploration should also give estimates or predictions of possible problem conditions which may be created by surface drainage, ground water, frost action, or the underlying bedrock. To keep the number of samples for testing to a minimum, the exploration includes field identification and soil classification. The samples that appear to be similar are carefully examined and only a representative selection is prepared for further detailed testing.

d. Soils are exposed for study by examining excavations, road cuts, and so on, and by making borings or digging test pits. The program for location and depth of pits and borings should insure that each type of soil in the area of interest is identified. Use is made of the information available in *a* above, as well as field reconnaissance, to locate preliminary borings where different soils occur. Even stations along the centerline survey are a convenient interval for borings for the beginning, to be modified by increasing the interval where conditions are uniform or by adding intermediate sites where the character of the soil profile changes. In cut and borrow pit areas, borings should extend 4 feet below the final subgrade or excavation depth. In fill areas they should extend 4 feet below the ground surface. Borings must extend to the depth of maximum frost. Test pits may be required where there is a need for samples of undisturbed soil. More detailed requirements for soil exploration are given in TM 5-330.

e. The soil exploration program must provide adequate field observations, complete and accurate notes locating the samples in plan and elevation, and a group of samples that are properly taken, handled, and identified. Failure to comply with even one of these conditions can destroy the value of the exploration. It may also result in misleading information and subsequent problems in future operations.

2-8. Field Identification

Lack of time and facilities often makes laboratory soil testing impossible in military construction. Even where laboratory tests are to follow, field identification tests can reduce the number of laboratory test samples to a minumum.

a. Preparation. Several simple tests are used in field identification. The number of tests depends on the type of soil and the experience of the individual employing them. Experience is the greatest asset in field identification, and learning the technique from an experienced technician is the best method of acquiring experience. Lacking such assistance, experience is gained by getting the "feel" of soils during laboratory testing. An approximate identification can be made by spreading a dry sample on a flat surface and examining it. All lumps should be pulverized until individual grains are exposed, but not broken, since this will change the grain size and the character of the soil. A rubber-faced or wooden pestle and a mixing bowl are recommended, but mashing underfoot on a smooth surface will suffice for an approximate identification. Examination of the characteristics of the particles in the sample makes it possible to assign the soil to one of the three principal groups (coarse-grained, fine-grained, or organic soils). Classifications derived from these tests should be recognized as approximations.

(1) An approximate identification of a coarse-grained soil is made by observing grain size, gradation, grain shape, and hardness of particles.

(2) Tests for identification of the fine-grained portion of any soil are performed on the portion of the material which passes a No. 40 sieve. This is the same soil fraction used in the laboratory for Atterberg limits tests, such as plasticity. If this sieve is not available, a rough separation may be made by spreading the material on a flat surface and removing the gravel and larger sand particles. Fine-grained soils are examined primarily for characteristics related to plasticity.

(3) Organic soils are those which contain significant quantities of organic matter. The presence of decayed roots, leaves, grasses, and other vegetable matter produces highly organic soil, which is usually dark in color when moist and has a soft spongy feel and a distinctive odor of rotting organic matter. Partly organic soils may contain finely divided organic matter detectable by color or odor.

b. Equipment Required. Field tests may be performed with little or no equipment other than a small amount of water. However, accuracy and uniformity of results will be greatly increased by the proper use of certain items of equipment available in nearly all engineer units. The equipment should include:

(1) *Sieves.* At least the No. 40, and possibly the No. 4 and the No. 200.

(2) *Pioneer tools.* A hand earth auger or post hole digger is useful to secure samples from depths a few feet or more below the surface.

(3) *Stirrer.*

(4) *Knife.*

(5) *Mixing bowl and pestle.*

(6) *Paper.*

(7) *Pan and heating element.*

(8) *Scales or balances.*

c. Factors Considered. The Unified Soil Classification System (para 2-3) considers three soil properties: percentage of gravel, sand, or fines; the shape of the grain-size distribution curve; and the plasticity. These are the primary ones to be considered, but other observed properties should also be included in the soil description whether made in the field or in the laboratory. The following information can be invaluable in the design phases:

(1) Color.

(2) Grain size.

(*a*) Estimated maximum grain size.

(*b*) Estimated percent by weight of fines (material passing No. 200 sieve).

(3) Gradation.

(4) Grain shape.

(5) Plasticity.

(6) Predominant soil type.

(7) Secondary components.

(8) Classification (symbol).

(9) Other, where applicable.

(*a*) Organic, chemical, or metallic content.

(*b*) Compactness (dense or loose).

(*c*) Consistency.

(*d*) Cohesiveness (ability to hold together as a plastic).

(*e*) Dry strength.

(*f*) Source (residual or transported).

d. Example of Soil Description. A complete description with the proper classification symbol obviously conveys much more to the user of the data than the symbol of any isolated portion of the description alone. An example of a soil description using the sequence and considering the properties referred to in *c* above might be as follows:

(1) Dark brown to white.

(2) Coarse grained, maximum particle size 2¾ inches, estimated—60 percent gravel, 36 percent sand, and 4 percent fines passing through No. 200 sieve.

(3) Poorly graded (gap-graded, insufficient fine gravel).

(4) Gravel particle subrounded to rounded.

(5) Nonplastic.

(6) Predominantly gravel.

(7) With considerable sand and a small amount of nonplastic fines (silt).

(8) GP.

(9) Slightly calcareous, no dry strength, dense in the undisturbed state.

2–9. Tests for Field Classification

Some or all of the following tests produce observations that pertain to the Unified Soil Classification System (table B–1). Their use permits field identification according to the outline in paragraph 2–8 as well as classification. Such tests as are appropriate to the given soil sample should be made. Some tests appear to yield duplicate results. The purpose of these tests is to get the best possible identification in the field. Thus, if a simple visual examination will define the soil type, only one or two of the other tests have to be made to verify or check the identification. On the other hand, when the result from a test is inconclusive, some of the similar tests should be tried to establish the best identification.

a. Visual Examination. This test should establish the color, the grain sizes, the grain shapes (of the coarse-grained portion), some idea of the gradation, and some properties of the undisturbed soil.

(1) *Color.* Color is often helpful in distinguishing between soil types, and with experience, may be useful in identifying the particular soil type. Color may also indicate the presence of certain chemicals or impurities. Color often varies with the moisture content of a soil. For this reason, the moisture content at the time of color identification should be included. Some of the more familiar color properties are stated below. Colors in general become darker as the moisture content increases and lighter as the soil dries. Some fine-grained soil (OL, OH) with dark, drab shades of brown or gray, including almost black, contain organic colloidal matter. In contrast, clean, bright looking shades of gray, olive green, brown, red, yellow, and white are associated with inorganic soils. Gray-blue or gray-and-yellow mottled colors frequently result from poor drainage. Red, yellow, and yellowish-brown result from the presence of iron oxides. White to pink may indicate considerable silica, calcium carbonate, or aluminum compounds.

(2) *Grain size.* The maximum particle size of each sample considered should always be estimated if not measured. This establishes the upper limit of the gradation curve. Gravels range down to the size of peas. Sands start just below this size and decrease until the individual grains are just distinguishable by the naked eye. The eye can normally see individual grains about 0.07-mm in size, or about the size of the No. 200 screen. Thus, silt and clay particles (which are smaller than this dimension) are indistinguishable as individual particles.

(3) *Grain shape.* While examining the sample for grain sizes, the shapes of the visible particles can be determined. Sharp edges and flat surfaces indicate angular shape while smooth, curved surfaces are associated with the rounded shape. Particles may not be completely angular nor completely rounded. These particles are called subangular or subrounded depending on which shape predominates.

(4) *Distribution of grain sizes.* Laboratory analysis must be performed when accurate distribution is to be determined. However, an approximation can be made during the visual examination.

(*a*) Separate the larger grains (gravel and some sand particles) from the remainder of the soil by picking them out individually.

(*b*) Examine the remainder of the soil and estimate the proportion of visible individual particles (larger than No. 200 sieve) and the fines (smaller than No. 200 sieve).

(*c*) Convert these estimates into percentages by weight of the total sample. If the fines exceed 50 percent, the soil is considered fine grained (M, C, or O); if the coarse material excedes 50 percent, the soil is coarse grained (G or S).

(*d*) Examine coarse-grained soil for gradation of particle sizes from the largest to the smallest. A good distribution of all sizes without too much or too little of any one size means the soil is well graded (W). Overabundance or lack of any size means the material is poorly graded (P).

(*e*) Estimate the percentage of the fine-grained portion of the coarse grained soil. If less than 5 percent (nonplastic fines) of the total, the soil may be classified either as a GW, GP, SW, or SP type, depending on the other information noted above.

(*f*) If the fine-grained portion ((*e*) above) exceeds 12 percent, the soil will be either silty (M) or clayey (C) and requires further testing to identify.

(*g*) Fine-grained portions ((*e*) above) between 5 and 12 percent (nonplastic fines or fines not interfering with free drainage, or 0–12 percent plastic fines) total are borderline and require double symbols such as GW–GM, SW–SM, etc.

(*h*) Fine-grained soils (M, C, or O) from (*c*) above, require other tests to distinguish further. Grain-size distribution of fine portions is not normally performed in field identification. However, should it become necessary, an approximation can be made by shaking the fine portions in a jar of water and allowing the material to settle. The materials will settle in layers of different sizes from which the proportion can be estimated. It should be kept in mind that gravel and sand settle into a much denser mass than either clay or silt. A description of the technique is given in paragraph 2–10.

(5) *Undisturbed soil properties.* Using characteristics determined up to this point, it is possible to evaluate the soil as it appeared in place. Gravels or sands can be described qualitatively as "loose", "medium", or "dense". Clays may be "hard", "stiff", or "soft". The ease or difficulty with which the sample was removed from the ground is a good indicator. Soils which have been cultivated or farmed can be further evaluated as "loose" and "compressible." Highly organic soils can be "spongy" and "elastic." In addition, moisture content of the soil influences the inplace characteristics. This condition should be recognized and reported with the undisturbed soil properties.

b. Breaking or Dry Strength Test. This test is performed only on the material passing the No. 40 sieve. It is used to measure the cohesive and plastic characteristics of the soil. The test distinguishes between the clayey (C) and the silty (M) soils.

(1) Separate the selected sample of the soil on the No. 40 sieve.

(2) Using a representative portion of the sample which has passed through the sieve, prepare a pat of soil about 1½ inches in diameter and ¼–½ inch thick by molding it in a wet, plastic state. Natural samples may be found in pats which are of the proper size, but may yield incorrect results. This is due to the variations in the natural drying and compaction processes. If natural samples are used, the results must be treated as approximations and verified later.

(3) Allow the pat to dry completely.

(4) Grasp the pat between the thumbs and forefingers of both hands and attempt to break it. See figure 2–3 for proper way to hold the pat.

(5) If the pat breaks, try to powder it by rubbing it between the thumb and forefinger of one hand.

(6) Typical results and the indicated classifications are as follows:

(*a*) Pat cannot be broken nor powdered by finger pressure—Very highly plastic soil (CH).

(*b*) Pat can be broken with great effort, but cannot be powdered—Highly plastic soil (CH).

(*c*) Pat can be broken, and powdered but with some effort—Medium plastic soil (CL).

(*d*) Pat breaks quite easily and powders readily—Slightly plastic soil (ML, MH, or CL).

(*e*) Pat has little or no dry strength and crumbles or powders when picked up—Nonplastic soil (ML or MH), or (OL or OH).

(7) The test described above is one of the best for distinguishing between plastic clays and nonplastic silts or fine sands. However, a word of caution is appropriate. Dry pats of highly plastic clays quite often display shrinkage cracks. To break the sample along such a crack will give an indication of only a very small part of the true dry strength of the soil. It is important to distinguish between a break along such a crack, and a clean, fresh break that indicates the true dry strength of the soil.

c. Roll or Thread Test. This test is performed only on the material passing the No. 40 sieve. A representative portion of the sample is mixed with water until it can be molded or shaped without sticking to the

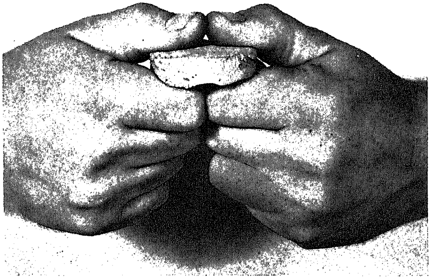


Figure 2–3. Breaking or dry strength test.

fingers. This moisture content is referred to as being just below the "sticky" limit.

(1) Prepare a nonabsorbent rolling surface by placing a sheet of glass or heavy wax paper on a flat or level support.

(2) Shape the sample into an elongated cylinder.

(3) Roll the prepared soil cylinder on the surface rather rapidly into a thread approximately 1/8 inch in diameter. The technique is shown in figure 2–4. If the moist soil rolls into a thread, it is said to have some plasticity. The number of times it can be rolled into a thread without crumbling is a measure of the degree of plasticity of the soil. Materials which cannot be rolled in this manner are nonplastic or have a very low plasticity.

(4) The results of this test indicate the following:

(*a*) Soil may be molded into a ball or cylinder and deformed under very firm finger pressure without crumbling or cracking—High plasticity (CH).

(*b*) Soil may be molded but cracks or crumbles under finger pressure—Medium plasticity (CL).

(*c*) Soil cannot be lumped into a ball or cylinder without breaking up—Low plasticity (CL, ML, or MH).

(*d*) Soil forms soft, spongy ball or thread when molded—Organic material (OL or OH), also peat.

(*e*) Soil cannot be rolled into a thread at any moisture content—Nonplastic soil (ML or MH).

NOTE

Micaceous silts and sands can roll due to the flaky nature of the mica. The wet shaking test is the only way to distinguish this property.

(5) From this test, the cohesiveness of the material near the plastic limit may also be described as weak, firm, or tough. The higher the position of a soil on the plasticity chart (table B–1), the stiffer are the threads as they dry out and the tougher are the lumps if the soil is remolded after rolling.

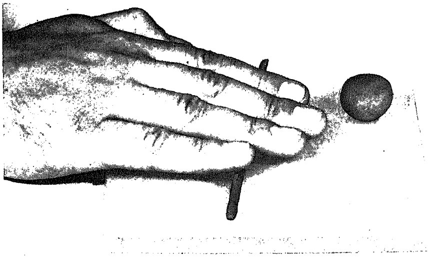


Figure 2–4. Roll or thread test.

d. Ribbon Test. This test is also performed only on material passing the No. 40 sieve. The sample is prepared as for the roll test (*c* above) until the moisture content is just below the sticky limit. This test and the roll test complement each other and give a clearer picture of the soil.

(1) Form a roll of soil about 1/2 to 3/4 of an inch in diameter and 3 to 5 inches long, or even longer.

(2) Lay the roll across the palm of one hand (palm up), and starting at one end, squeeze it between the thumb and forefinger over the edge of the hand to form a flat unbroken ribbon about 1/8 to 1/4 inch thick. As formed, the ribbon is allowed to hang free and unsupported (fig. 2–5).

(3) Continue squeezing and handling carefully to form the maximum length of ribbon that can be supported only by the cohesive properties of the soil.

(4) The typical results and their indication are as follows:

(*a*) Sample holds together for a length of 6 to 10 inches without breaking—Highly plastic and highly compressive (CH).

(*b*) Soil cannot be ribboned—Nonplastic (ML or MH).

(*c*) Soil can be ribboned only with difficulty into short lengths—Low plasticity (CL).

e. Wet Shaking Test.

(1) *Sample preparation.* The wet shaking test is performed only on the material passing the No. 40 sieve. For this test, enough material to form a ball of material about 3/4 inch in diameter is moistened with water. This sample should be just wet enough that the soil will not stick to the fingers upon remolding or just below the "sticky limit".

(2) *Testing.* The sample is then placed in the palm of the hand and shaken vigorously. This is usually done by jarring the hand on the table or some other firm object, or by jarring it against the other hand. The soil is said to have given a reaction to this test when, on shaking, water comes to the surface of the sample producing a smooth, shiny appearance. This appearance is frequently described as "livery" (1 and 2, fig. 2–6). Then, upon squeezing the sample between the thumb and forefinger of the other hand, this surface water will quickly disappear, the surface will become dull (3, fig. 2–6), the material will become firm, resisting deformation, and cracks will occur as pressure is continued, with the sample finally crumbling like a brittle material (4, fig. 2–6). The vibration caused by the shaking of the soil sample tends to reorient the soil grains, decrease the voids, and force water, which had been within these voids, to the surface. Pressing the sample between the fingers tends to disarrange the soil grains and increase the voids space, and the water is drawn into the soil. If the water content is still adequate, shaking the broken pieces will cause them to liquefy again and flow together, and the complete cycle may be re-

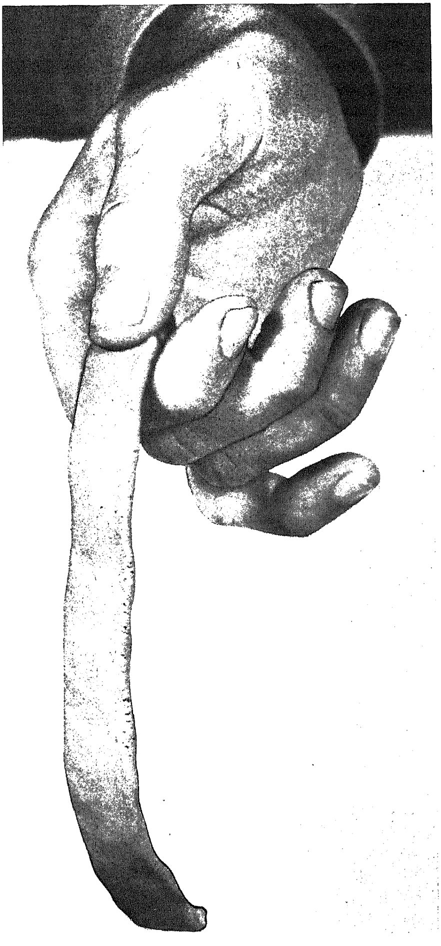


Figure 2-5. Ribbon test.

peated. This process can occur only when the soil grains are bulky in shape and noncohesive in character.

(3) *Typical reactions.* Very fine sands and silts fall into this category and are readily identified by the wet shaking test. Since it is rare that fine sands and silts occur without some amount of clay mixed with them, there are varying degrees of reaction to this test. Even a small amount of clay will tend to greatly retard this reaction. Some of the descriptive terms applied to the different rates of reaction to this test are as follows:

(*a*) *Sudden or rapid.* A rapid reaction to the shaking test is typical of nonplastic, fine sands and silts (2, fig. 2-6).

(*b*) *Sluggish or slow.* A sluggish reaction indicates slight plasticity such as might be found from a test of some organic silts, or silts containing a small amount of clay.

(*c*) *No reaction.* Obtaining no reaction at all to this test does not indicate a complete absence of silt or fine sand.

f. Odor Test. Organic soils (OL and OH) usually have a distinctive, musty, slightly offensive odor. With experience, the odor can be used as an aid to identifying such materials. This odor is especially apparent from fresh samples, but becomes less pronounced as the sample is exposed to air. It can be made stronger by heating a wet sample.

g. Bite or Grit Test. This test is a quick and useful one in identifying sand, silt, or clay. A small pinch of

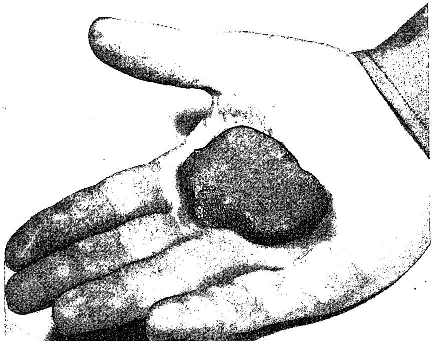


Figure 2-6. Wet shaking test. (sheet 1) Initial sample.

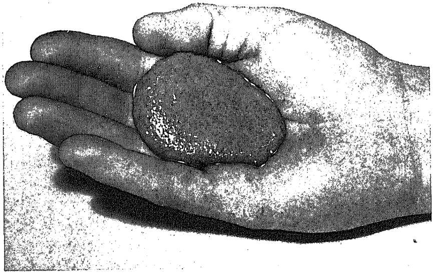


Figure 2-6. Wet shaking test. (sheet 2) Livery appearance.

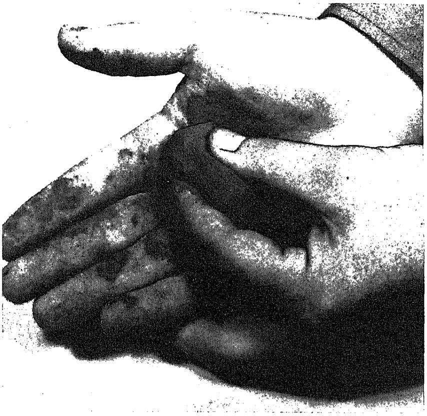


Figure 2–6. Wet shaking test (sheet 3) Squeezing the sample.

soil is ground lightly between the teeth. The typical reactions of the different types of soil are as follows:

(1) *Sandy soils.* The sharp hard particles of sand will grate very harshly between the teeth and will be highly objectionable. This is true even of the fine sands.

(2) *Silty soils.* Silt grains are so much smaller than sand grains that they do not feel nearly as harsh between the teeth. They are not particularly gritty although their presence is still quite unpleasant and easily detected.

(3) *Clayey soils.* Clay grains are not gritty, but feel smooth and powdery like flour between the teeth. Dry lumps of clayey soils will stick when lightly touched with the tongue.

h. Slaking Test. This test assists in determining the quality of some soft shales and other "rocklike" materials. Place a sample of the soil in the sun or in an oven to dry. Then, place it in a bucket of water and soak it for at least 24 hours. Examine the strength of the soil after this period. Certain types of shale will completely disintegrate and lose all strength after this treatment.

i. Acid Test. The acid test is performed to determine the presence of calcium carbonate and requires hydrochloric acid. Place a few drops of acid on a piece of the soil. A fizzing reaction (effervescence) indicates the presence of calcium carbonate and the degree of reaction indicates the concentration. In very dry soils, the absorption of the acid creates an illusion of effervescence. This can be eliminated in all dry soils by moistening the soil prior to applying the acid. Calcium carbonate provides a degree of cementing action in a soil. This test permits an understanding of what appears to be abnormally high strength in a fine grained soil tested in place.

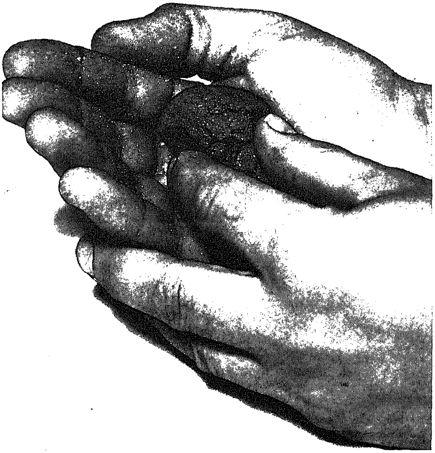


Figure 2–6. Wet shaking test (sheet 4) Crumbling the sample.

j. Shine Test. This test is another means of determining plasticity characteristics of a soil. A slightly moist or dry piece of highly plastic clay will produce a definite shine when rubbed with a fingernail or a smooth metal surface, such as a knife blade. Lean clay will remain dull after this treatment.

k. Feel Test. This is a general purpose test and requires considerable experience and practice before reliable results can be expected. The extent of its use will increase with increasing familiarity with soils. Consistency and texture are two characteristics which can be determined.

(1) *Consistency.* The natural moisture content is an indicator of the soil drainage which may affect this characteristic.

(*a*) For the test, squeeze a piece of undisturbed soil between the thumb and forefinger to determine its consistency. Description of consistency is made by such terms as "hard," "stiff," "brittle," "friable," "sticky," "plastic," and "soft".

(*b*) Remold the soil working it between the hands, and observe the results. This can be an indicator of the natural water content. Clays which become fluid on remolding are probably near their liquid limit. If they remain stiff and crumble when reworked, they are probably below the plastic limit.

(2) *Texture.* This term is applied to the fine grained portion of the soil, and refers to the degree of fineness and uniformity.

(*a*) Rub a portion of the soil between the fingers.

(*b*) Observe the texture and describe it as "floury", "smooth", "gritty", or "sharp".

(*c*) Sensitivity may be increased by rubbing the soil on a more tender skin area as the inside of the wrist.

(*d*) Typical results are somewhat similar to the bite test. Sand will feel gritty. Silts, if dry, will dust readily and feel soft and silky to the touch. Clays powder only with difficulty, but feel smooth and gritless like flour.

2–10. Hasty Methods of Testing

With the standard methods of field identification supplemented with a few simplified tests, an approximate and hasty classification can be obtained of almost any soil. The three simple or hasty tests outlined below will, for the most part, eliminate the need for specialized equipment such as sieves. The results of these tests, when used or supplemented with the results of the tests described in the previous paragraph, will give at least a tentative classification to almost any soil. The schematic diagram (fig. 2–7) may be used as a guide to the testing sequence in the process of assigning a symbol to a sample soil.

a. Sedimentation Test. From the visual observation test (para 2–9*a*) it is relatively simple to approximate the proportions of sand and gravel in a soil by spreading the dry sample out on a flat surface, and separating the gravel particles by hand. Separating the fines from the sand particles is more difficult although just as important. Smaller particles will settle through water at a slower rate than large particles. By placing a small representative amount of the fine fraction of soil (such as a heaping tablespoon measure) in a transparent cup or jar, covering with about 5 inches of water, and agitating by stirring or shaking, the soil will be completely suspended in water. With most cohesive soil it will be necessary to break up the lumps of soil before adding the water. This can be done by grinding the soil in a canteen cup with an improvised wood pestle. After the soil particles have been thoroughly dispersed in the water and then left, they will start to settle out, beginning with the larger size particles, in time periods approximately as indicated below:

Approximate time of settlement through 5 inches of water	*Grain diameter (mm)*	*Differentiates*
2 seconds	0. 4	Coarse sand—fine sand.
30 seconds	0. 072	Sand—fines.
10 minutes	0. 03	Coarse silt—fine silt.
1 hour	0. 01	Silt—clay.

(1) The most important use of the sedimentation test is to differentiate the coarse (0.072-mm) fraction from the fine fraction of a soil. Since all of the particles of soil larger than 0.072-mm will have settled to the bottom of the cup or jar 30 seconds after the mixture has been agitated, it follows that the particles still remaining in suspension are fines. If the water containing the suspended fines is carefully poured into another container 30 seconds after agitation, if more water is added to the cup or jar containing the coarse fraction, and if the procedure is repeated until the water-soil mixture becomes clear 30 seconds after mixing, then the cup or jar will contain the coarse fraction of soil and the pan containing the suspension will hold the fines. If the water can be wicked or evaporated off, the relative amounts of fines and sand can be determined fairly accurately. Otherwise a direct measurement of the settled out fines can be obtained as a guide. Thus, in a sense, the test acts like a No. 200 sieve.

(2) Most field identification tests are performed on the No 40 sieve soil fraction (fines and fine sand portion). This fraction can be separated by using a procedure similar to that outlined above, except that the water is poured off within 1 or 2 seconds after completion of agitation. The suspended portion will then include the particles of the fine sand range.

(3) A difficulty that will be encountered with many clay soils stems from the fact that the clay particles will often form small lumps (flocculate) that will not break up in water. Usually this condition can be detected by examining the coarse fraction of the soil after several repetitions of the test. If substantial amounts of clay are still present, the sand will have a somewhat slippery feel and further mixing and grinding with a wood stick will be necessary to help break up these lumps.

b. Cast Test. The cast test refers to the strength of a moist soil sample when squeezed in the hand. It is used to indicate the approximate type and quantity of fines present in the sample. The correct amount of water to add to the soil must be estimated by trial and error, although it can generally be stated that maximum cohesion or attraction between the individual soil particles normally will occur when the soil is damp but not sticky. The test consists of compressing a handful of the moist soil into a ball or cigar-shaped cast and observing its ability to withstand handling without crumbling. While experience is desirable in making predictions based upon this test, the listing below may serve as a general guide for the behavior of different soil types when formed into a cast and tested:

Soil type	*Reaction to handling*
GP, SP, SW, GW	Cast crumbles when touched.
SM, SC	Cast withstands careful handling.
ML, MH	Cast can be handled freely.
CL, CH	Cast withstands rough handling.

c. Wash, Dust, and Smear Tests. A small amount of silt (less than 5 percent), when intermixed with a coarse-grained soil, normally will not lessen the value of the soil as a construction material. However, increasing quantities of silt will sharply reduce the

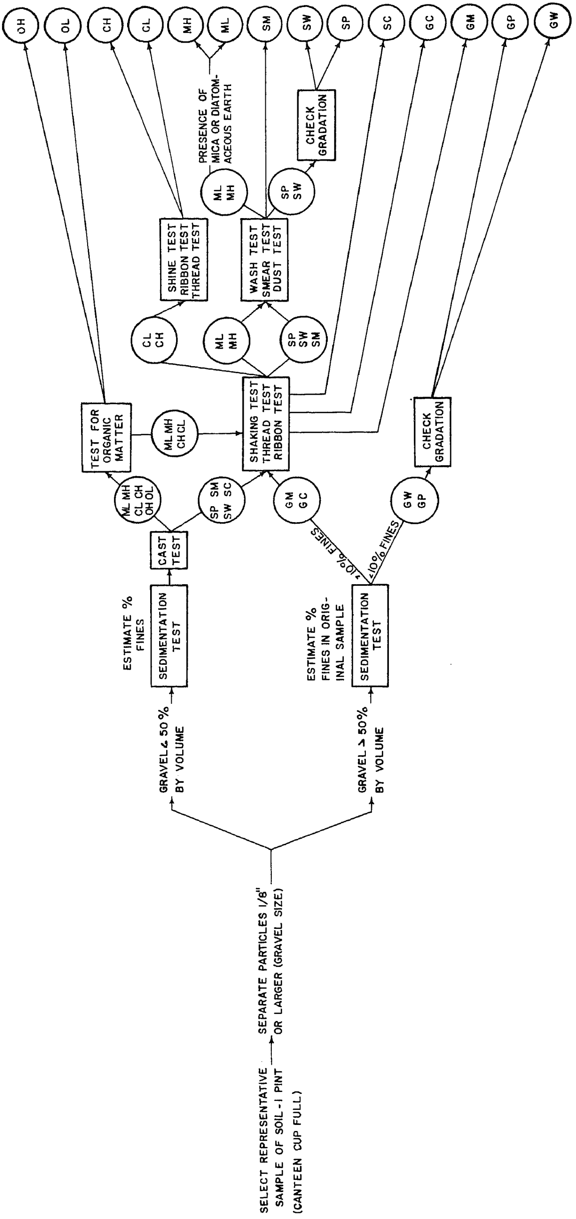


Figure 2–7. Suggested procedure for hasty field identification.

strength and interfere with the free drainage characteristics of the coarse-grained soil, thus making it less desirable as a road or airfield construction material. To decide what constitutes a harmful concentration of silt by the field identification methods generally requires extensive field experience. If the dust, wash, and smear tests are first practiced on soils of known silt contents, they can be used to produce a fairly accurate result. In the dust test, when a completely dry sample of soil (with the gravel portion removed) is dropped from a height of 1 or 2 feet onto a solid surface, a silt content higher than 10 percent will generally cause a fairly large amount of dust to be produced. In the wash test, an identical soil sample, as above, is placed in the palm of a hand, and covered with about ⅛ inch of water. If the water becomes completely discolored and hides the sand grains, this indicates that the soil sample contains a silt content higher than 5 percent. In the smear test, a sample of soil, again with the gravel portion removed, is moistened to just below the "sticky limit", and then smeared between the thumb and forefinger. When it produces a gritty, harsh feel, this indicates that it contains a silt content of less than 10 percent. A rough, less harsh feel, however, indicates that the sample contains more than 10 percent silt. It should be emphasized that all of the above tests require the testing engineer to have some experience in their use before they can be considered as reliable indicators of the silt contents.

Section II. SAMPLING

2-11. Sample Selection

a. The purposes of soil surveys and field identification were discussed in previous paragraphs. Representative soil samples are taken during these operations for field identification and for more detailed testing. For hasty investigations, samples from the surface or from excavations already in existence may be sufficient. The exploratory survey and field identification, together with the requirements of the construction, dictate the location, depth, type of sample needed and sampling method to be used.

b. The two most commonly used methods for obtaining samples are the test hole and the test pit. Samples from test holes, or borings, are taken by hand or power augers, wash borings, dry sample borings, undisturbed sampling, or core borings. Augers and other borings yield disturbed samples satisfactory for determining the soil profile, classification, moisture content, compaction capabilities, and similar properties. Special sampling tools are needed to take undisturbed samples from test holes. Test pits are open excavations large enough for a man to enter and study the soil in its undisturbed condition. They may be dug by hand or by power equipment if available. Undisturbed samples are taken from the walls of the test pits. Load bearing tests can be made, if desired, on the soil in the bottom of the pit.

2-12. Locating, Recording, and Numbering Samples

a. Locating. Information obtained from each test pit, boring, or other exploratory excavation must be correlated with data obtained from other sources. In exploration for roads and airfields, locations are established by measurements along the centerline or at right-angle offsets to the centerline, as required. Measurements may be made by pacing or taping. The soil testing set contains a 100-foot steel tape for this purpose. The location, type, and identification of the exploratory excavation are plotted on a photograph, map, or sketch made in the notebook as illustrated in figure 2-8. A uniform system should be used for plotting exploratory excavations. The symbols below are in common use.

Type	*Abbreviation*	*Symbol*
Pit	P	Square.
Boring	B	Circle.
Trench	T	Rectangle.

b. Recording. A detailed record or log of each exploratory excavation (test pit) is kept in a fieldbook as shown in figure 2-9. Logs must be complete and contain all information which may be of value in interpreting the test results and in plotting the soil profiles. Interpretation of the log will be made easier if each of the soils encountered is identified in the field and classified under the Unified Soil Classification System. Soil descriptions must be accurate and complete (para 2-8).

c. Numbering Samples. Each sample is identified by the following information:

(1) A project number which consists of a number, an abbreviation, or a symbol.
For example, BF might be used for Benton Field.

(2) An excavation number which corresponds to the number of the exploratory excavation.
Fcr example, P3 would mean that the sample was obtained from test pit number 3.

(3) A sample number, which is the number of the sample in the order in which it was obtained from each location. If more than one bag sample is taken from one location, the total number of bags comprising the sample also must be recorded. An example of a complete designation number is as follows:

B10-P3-2
(Bag 1 of 2)

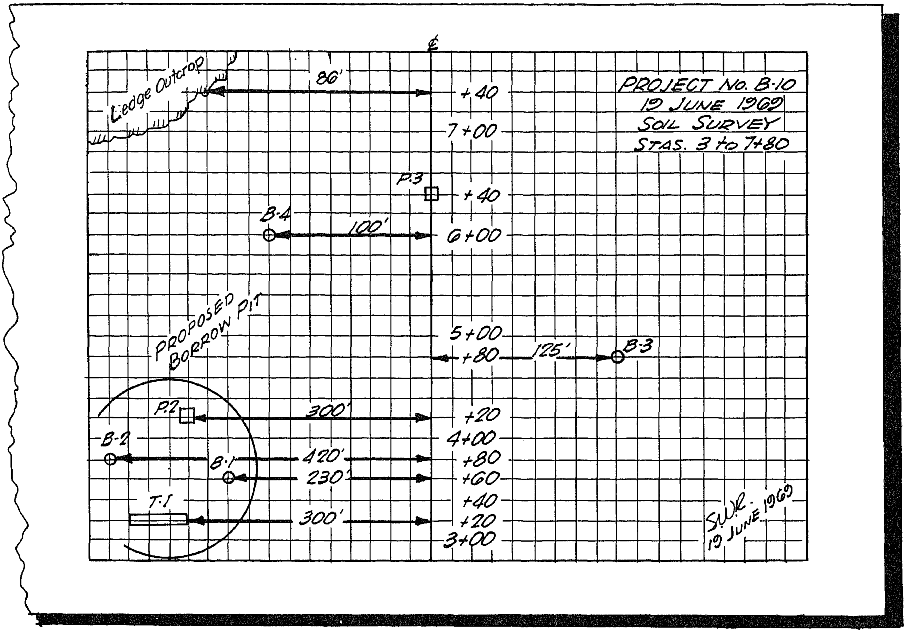


Figure 2–8. Typical fieldbook page showing locations of soil explorations.

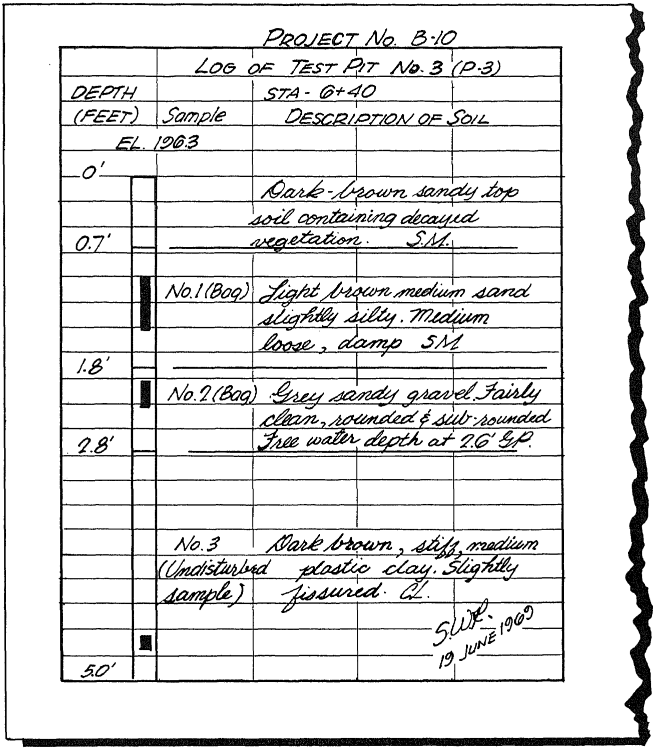
This designation means that the sample was taken on project B10, pit number 3, that it is the second sample taken in this pit, and that the bag is the first of a total of two bags. The sample number is printed with a glazed surface-marking pencil on two tags, one of which is placed inside the bag, and the other of which is tied on the outside, as shown in figure 2–10. Gummed labels may be similarly used to mark samples which are contained in moisture content boxes, cylinders, or jars.

d. Responsibility. Responsibility of the officer in charge of soil exploration is to see that:

(1) Each test pit, boring, or other exploratory excavation is properly surveyed, numbered, and recorded.

(2) A log is kept of each test hole or test pit.

(3) Each sample is properly numbered, tagged, or otherwise labeled, and prepared for shipment or transportation to the laboratory.



PROJECT No. B-10

LOG OF TEST PIT No. 3 (P-3)

STA - 6+40

DEPTH (FEET)	Sample	DESCRIPTION OF SOIL
EL. 1963		
0'		Dark-brown sandy top soil containing decayed vegetation. S.M.
0.7'	No. 1 (Bag)	Light brown medium sand slightly silty. Medium loose, damp SM
1.8'	No. 2 (Bag)	Grey sandy gravel. Fairly clean, rounded & sub-rounded Free water depth at 2.6' GP.
2.8'	No. 3 (Undisturbed sample)	Dark brown, stiff, medium plastic clay. Slightly fissured. CL.
5.0'		

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Figure 2-9. Typical fieldbook page showing log of test pits.

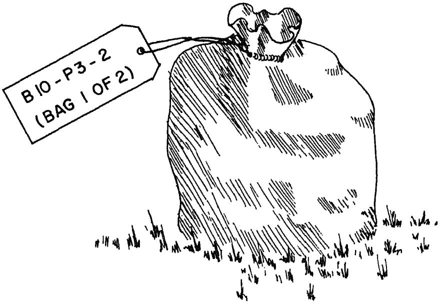


Figure 2-10. Method of labeling bag samples.

2-13. Bag Samples

Disturbed soil samples taken by means of a shovel, auger, or any other convenient handtool and placed in bags are known as bag samples. They are used for mechanical analysis, plasticity, specific gravity, frost susceptibility, compaction, and laboratory compacted CBR tests. The size of the sample taken will depend upon the tests to be conducted. Two sizes of bags are issued in the soil test set.

a. Individual Samples. When exploring foundation conditions, bag samples are taken of each individual type of soil encountered.

(1) To take individual samples from a pit, trench, or exposed face, shave off loose and dried soil

to obtain a fresh surface, and to expose clearly any variations in the soil. Then a typical sample of each type of soil, or of those types which require further investigation, is taken as illustrated in figure 2–11.

(2) When sampling in auger holes, typical portions of soil obtained are placed along a row in correct order as indicated in figure 2–12.

b. Composite Samples. A composite sample is a representative mixture of all soil within a profile or soil mass to be investigated, or of the material contained in a stockpile or windrow of soil excavated from a trench. A test sample is obtained from a composite sample by quartering in the laboratory (para 2–22)

(1) To take composite samples from test pits, trenches, or power shovel cuts, the following steps are required:

(*a*) Any overburden or surface soil that is to be wasted is removed.

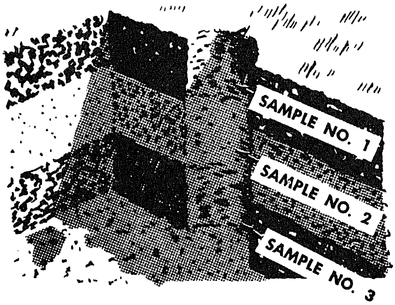


Figure 2–11. Obtaining individual bag samples from an exposed face.

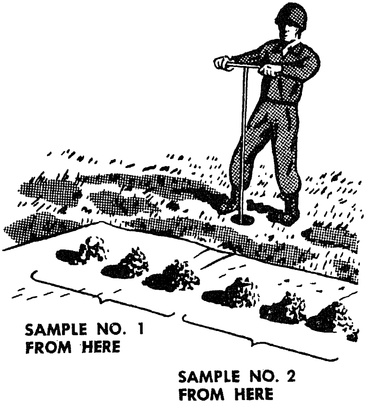


Figure 2–12. Obtaining individual bag samples from an auger boring.

(*b*) Loose and dried soil is shaved off to obtain a fresh surface for taking the sample.

(*c*) A quartering cloth or tarpaulin is spread at the toe of the bank.

(*d*) A channel of uniform cross section is excavated from top to bottom (fig. 2–13), and the soil deposited on the canvas.

(*e*) All material removed is collected and bagged to insure that the sample contains the correct proportions.

(2) To take composite samples from auger holes, all material excavated from the hole is collected, after first removing overburden.

(3) Particular care must be taken to obtain representative samples when composite samples are being taken from stockpiles or large windrows. When material is dumped on large piles, the coarse material tends to roll to the bottom, leaving the finer material in the upper portion. The sample is taken from a full-height strip after clearing the surface.

(4) To sample a small windrow, material is excavated and bagged from a short section, as illustrated in figure 2–14.

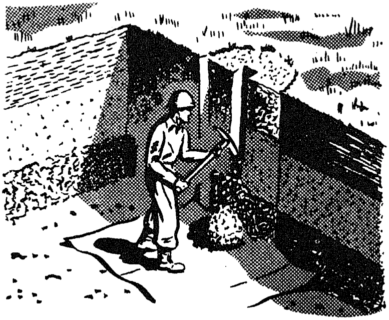


Figure 2–13. Taking a composite sample from an exposed face.

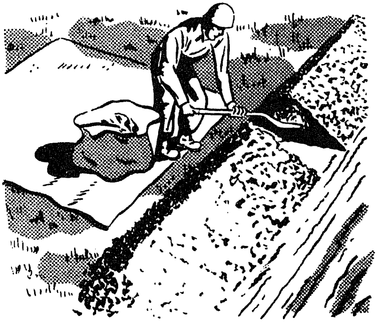


Figure 2–14. Taking a composite sample from a small windrow.

2-14. Moisture Content Samples

The natural moisture content of soil is determined from samples taken in the field and placed in a container which is sealed to prevent loss of moisture by evaporation. Natural moisture content determinations are valuable in interpreting information obtained from test borings or pits, in drawing the soil profile, and in estimating the physical properties of soils encountered in the field (para 2-2*f*). Generally, 100 grams of soil is enough to determine the moisture content of fine grained soils. Larger samples are required for soils which contain gravel. Three sizes of metal dishes are provided in the soil test set. These have tight-fitting covers and do not required sealing if the test is made within 1 day after the sample is taken. If a longer time interval will elapse between sampling and testing, the boxes may be sealed by the method illustrated in figure 2-15. Any other clean containers which can be adequately sealed may be used for moisture-content samples.

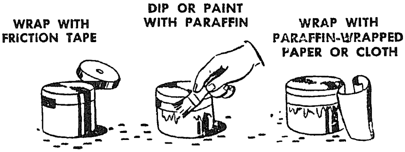

WRAP WITH FRICTION TAPE
DIP OR PAINT WITH PARAFFIN
WRAP WITH PARAFFIN-WRAPPED PAPER OR CLOTH

Figure 2-15. Sealing container in order to retain moisture content of a sample.

2-15. Undisturbed Samples

An undisturbed soil sample is one that is cut, removed, and packed with the least possible disturbance. They are samples in which the natural structures, void ration, and moisture content are preserved as carefully as possible. Samples of this type are used for determining the density (unit weight) of soil in the laboratory, and investigating the strength of undisturbed soils in the laboratory by the California bearing ratio or unconfined compression tests. These samples may be shipped to more completely equipped laboratories for shear, consolidation, or other strength tests.

a. Types of Undisturbed Samples. Equipment is available for obtaining undisturbed samples of three different types and by several methods as follows:

(1) Chunk samples cut by hand with shovel and knife.

(2) Cylinder samples obtained by use of a cylindrical sampler or the CBR mold equipped with sampling cutter. Expedient methods of obtaining cylinder samples are also used.

(3) Box samples.

b. Method. The method of sampling chosen depends upon the equipment available, the tests required, and type of soil. All undisturbed samples must be handled with care. Cohesionless soil samples must be kept in the container in which taken until ready for testing and the container should be handled without jarring or vibration. Some soils are too hard or contain too many stones to permit sampling with the cylindrical samplers and can be sampled only by cutting out chunks by hand. The taking of undisturbed samples frequently requires a great deal of ingenuity in adapting the sampling devices to job conditions and in devising schemes for their use. Whatever method is used, the sample must be taken and packed in the container for shipment without allowing its structure to change. Protection against change in moisture content during sampling and shipment is also required.

2-16. Chunk Samples

The simplest type of undisturbed sample is obtained by cutting out a chunk of soil of the desired size and covering it to prevent loss of moisture and breakage. This method can be used only with soils that will not deform, break, or crumble while being removed.

a. The process of obtaining a chunk sample from a subgrade or other level surface, such as the bottom of a test pit, is illustrated in figures 2-16, 2-17, and 2-18. The first step is to smooth the ground surface and mark the outline of the chunk. A trench is excavated around the chunk (fig. 2-16), after which the excavation is deepened, and the sides of the chunk trimmed with a knife (fig. 2-17). The chunk is then

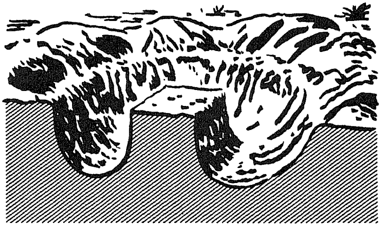


Figure 2-16. Taking chunk sample from a level surface. Step 1—excavate trench.

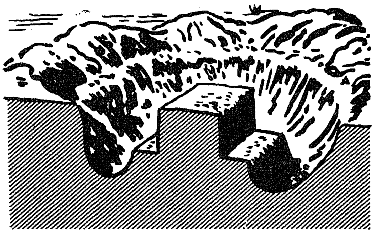


Figure 2-17. Taking chunk sample from a level surface. Step 2—trim sample.

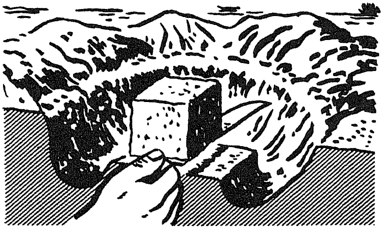


Figure 2–18. Taking chunk sample from a level surface. Step 3—cut off and remove.

cut off at the bottom with a knife, trowel, or hacksaw blade, and carefully removed from the hole (fig. 2–18).

b. To obtain a chunk sample from the vertical face of a test pit or trench (figs. 2–19, 2–20, and 2–21), the first step again is to carefully smooth the surface of the face and mark the outline of the chunk. Soil is excavated from around and in back of the chunk (fig. 2–19). The chunk is shaped with the knife (fig. 2–20), after which it is cut off and carefully removed from the hole (fig. 2–21).

c. The chunk sample must be sealed after removing it from the hole. One method is to apply three coats of melted paraffin, as illustrated in figure 2–22. Each coat is allowed to cool and become firm before applying the next coat. This gives adequate protection for strong samples that are to be used within a few days. Samples that are weak, or which may not be used soon, require additional protection. They should be wrapped with cheesecloth or other soft cloth (fig. 2–23) and sealed in paraffin. If cloth is not available, the sample can be reinforced with several loops of friction tape or twine. Three more coats of paraffin then are applied. Extreme care is required to prevent damage to the structure or other disturbance to the sample while performing these operations.

d. An alternate method of sealing a chunk sample is to dip the entire sample in melted paraffin (fig. 2–24) after the first brush coat has been applied and

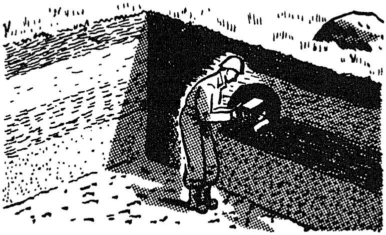


Figure 2–19. Taking a chunk sample from a vertical face. Step 1—excavate pit and sample tunnel.

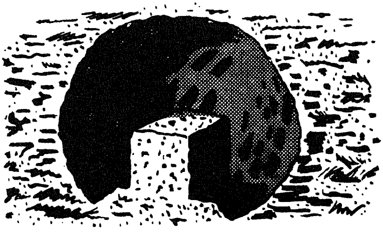


Figure 2–20. Taking a chunk sample from a vertical face. Step 2—trim sample.

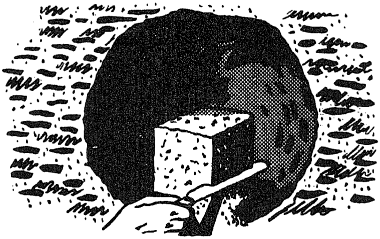


Figure 2–21. Taking a chunk sample from a vertical face. Step 3—cut off and remove.

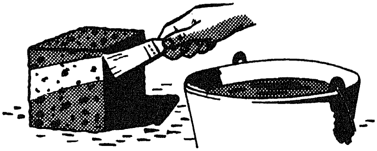


Figure 2–22. Applying paraffin to seal chunk sample.

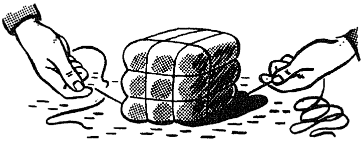


Figure 2–23. Wrapping a weak chunk sample prior to final sealing.

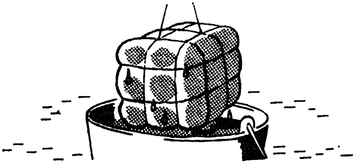


Figure 2–24. Dipping chunk sample in melted paraffin.

the sample wrapped. This requires a large container and more paraffin, but gives a more uniform coating. By repeated dipping, the layer of paraffin can be built up to a minimum ⅛ inch thickness.

e. Samples that are to be shipped require additional protection. This may be accomplished by placing the chunk in a small box and packing, as illustrated in figure 2–25, or by applying many coats of cloth and paraffin.

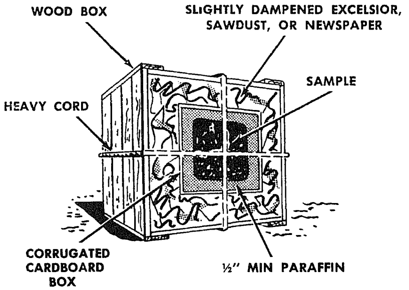


Figure 2–25. Packing a chunk sample for transportation or shipment to laboratory.

2–17. Cylinder Samples by Drive Sampler

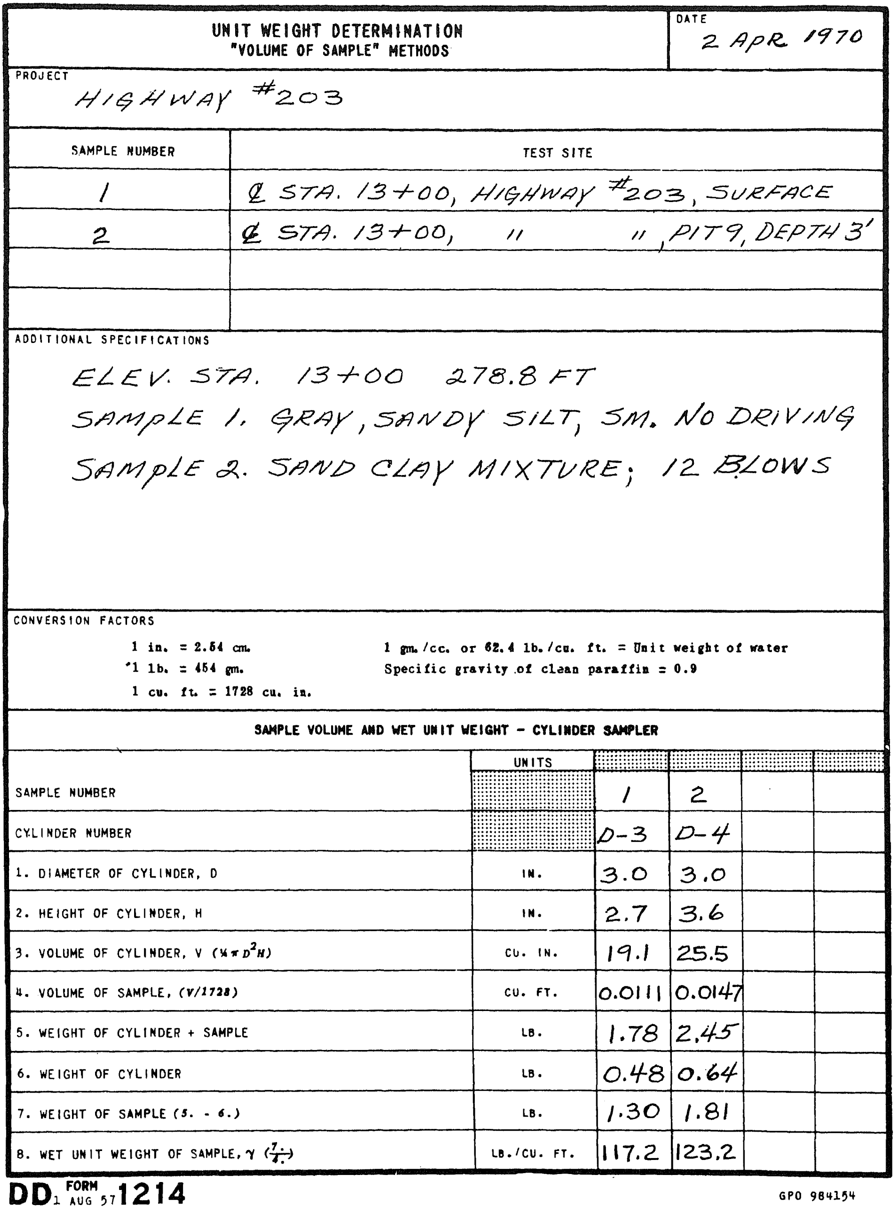
A drive sampler is a device used for obtaining relatively undisturbed soil samples suitable for laboratory tests at the surface or at the bottom of shallow borings or test holes, and at the bottom of test pits. It consists of a sampling tube or cylinder which is driven into the soil. One end of the tube is tapered to a cutting edge which penetrates the soil. The tube or cylinder is used in conjunction with a drive head and rod. Drive samplers' use is restricted to soils from soft to fairly firm, and free of stones or rocks which can damage the equipment. Drive samplers are not issued in the Soil Test Set but may still be available in some units.

a. Prepare data sheet, DD form 1214 (Unit Weight Determination "Volume of Sample" Methods) (fig. 2–26), recording date, project, test site location and number as described in paragraph 2–12, surface elevation, sample depth and number, type and size of sampler, method of driving sampler, and blows required. Description of soil and field identification corresponding to the sample should be included, if available.

b. Weigh the sampling cylinder, after cleaning to remove any adhering soil or corrosion, and record weight of cylinder as item 6, DD form 1214 (fig. 2–26). Measure and record diameter and height of cylinder. The drive head with the rod attached is placed over the sampling tube or cylinder. The cylinder is then "driven" into the soil using the hand-held hammer (fig. 2–28) or the drophammer built as part of the drive rod assembly (fig. 2–29). When the sampling cylinder is fully driven and some soil is seen above the top of the cylinder, the cylinder and soil sample are carefully dug out. The sample is carefully trimmed with a knife at both bottom and top, and weighed. Record this weight of sample and tube on line 5. The weight of the sample and its weight per unit volume (items 7 and 8) can now be computed. The reverse side of DD form 1214 (fig. 2–27) is used for weight per unit volume determination of paraffin coated samples and water content determinations which are discussed in section VIII.

c. Samples to be transported to the laboratory must be carefully sealed and protected. If field weighing (*b* above) is not possible, the sample must be sealed in the tube immediately after taking. If left in the cylinder, the ends must be sealed with a thin coat of paraffin. If the samples are to be handled much or removed from the cylinder, they should be carefully sealed, reinforced, and protected. Cloth or proper reinforcement should be used in conjunction with paraffin applied by use of paintbrush or dipping. The soil core is laid on a flat, clean board and the reinforcing paper or cloth is coated with paraffin as it is rolled onto the core sample. Sufficient reinforcing material is left at the ends to fold in to protect the ends of the core.

d. Undisturbed samples may be taken from greater depth in test holes by thin-walled tube sampling. The hole is made by a suitable method to the desired depth for taking the sample. The boring operation must not disturb the soil below the bottom of the hole. Thin-walled sample tubes, 2 to 5 inches in diameter, having lengths 5 to 10 times the diameter and of adequate strength and corrosion resistance, are used. A sharpened cutting edge is desirable, and the upper end must be fitted with a coupling to connect to pipe extensions on the drilling apparatus to drive the tube into the soil and withdraw it with the sample. The test hole must be clean of loose material and water should remain at the ground water level during sampling. The tube should be pushed into the ground by a continuous downward motion without twisting. If it is necessary to use a driving hammer in hard soils, the weight, height of drop, and number of blows should be recorded. The tube should then be turned through two revolutions to shear off the sample at the bottom of the tube. The ends of the sample tube are then plugged with impervious disks, sealed, and labeled for shipment to the laboratory.



UNIT WEIGHT DETERMINATION
"VOLUME OF SAMPLE" METHODS

DATE 2 APR 1970

PROJECT HIGHWAY #203

SAMPLE NUMBER	TEST SITE
1	℄ STA. 13+00, HIGHWAY #203, SURFACE
2	℄ STA. 13+00, " " , PIT 9, DEPTH 3'

ADDITIONAL SPECIFICATIONS

ELEV. STA. 13+00 278.8 FT

SAMPLE 1. GRAY, SANDY SILT, SM. NO DRIVING

SAMPLE 2. SAND CLAY MIXTURE; 12 BLOWS

CONVERSION FACTORS

1 in. = 2.54 cm.

1 lb. = 454 gm.

1 cu. ft. = 1728 cu. in.

1 gm./cc. or 62.4 lb./cu. ft. = Unit weight of water

Specific gravity of clean paraffin = 0.9

SAMPLE VOLUME AND WET UNIT WEIGHT - CYLINDER SAMPLER

	UNITS				
SAMPLE NUMBER		1	2		
CYLINDER NUMBER		D-3	D-4		
1. DIAMETER OF CYLINDER, D	IN.	3.0	3.0		
2. HEIGHT OF CYLINDER, H	IN.	2.7	3.6		
3. VOLUME OF CYLINDER, V ($\frac{1}{4}\pi D^2H$)	CU. IN.	19.1	25.5		
4. VOLUME OF SAMPLE, (V/1728)	CU. FT.	0.0111	0.0147		
5. WEIGHT OF CYLINDER + SAMPLE	LB.	1.78	2.45		
6. WEIGHT OF CYLINDER	LB.	0.48	0.64		
7. WEIGHT OF SAMPLE (5. - 6.)	LB.	1.30	1.81		
8. WET UNIT WEIGHT OF SAMPLE, γ ($\frac{7.}{4.}$)	LB./CU. FT.	117.2	123.2		

DD FORM 1 AUG 57 1214

GPO 984154

Figure 2-26. Data sheet, unit weight determination by volume of sample (DD form 1214).

SAMPLE VOLUME AND WET UNIT WEIGHT – PARAFFIN COATED SAMPLES

	UNITS				
SAMPLE NUMBER		856A	862C		
9. WEIGHT OF SAMPLE + PARAFFIN IN AIR	GM.	1250.6	1851.1		
10. WEIGHT OF SAMPLE IN AIR	GM.	1200.3	1275.9		
11. WEIGHT OF PARAFFIN IN AIR *(9.-10.)*	GM.	50.3	75.2		
12. VOLUME OF PARAFFIN $\left(\frac{11.}{sp.\ gr.\ of\ paraffin}\right)$	CC.	55.9	83.6		
13. WEIGHT OF SAMPLE + PARAFFIN IN WATER	GM.	600.0	675.7		
14. VOLUME OF SAMPLE + PARAFFIN *(9.-13.)*	CC.	650.6	675.4		
15. VOLUME OF SAMPLE *(14.-12.)*	CC.	594.7	591.8		
16. WET UNIT WEIGHT OF SAMPLE, γ $\left(\frac{10.}{15.} \times 62.4\right)$	LB./CU. FT.	125.4	117.3		

WATER CONTENT DETERMINATION

	UNITS								
TARE NUMBER		27	28	29	30				
17. WEIGHT WET SOIL & TARE	gm	37.40	40.92	56.30	55.20				
18. WEIGHT DRY SOIL & TARE	gm	35.54	38.15	55.90	54.87				
19. WEIGHT OF WATER *(17.-18.)*	gm	1.86	2.77	0.40	0.33				
20. WEIGHT TARE	gm	31.31	31.88	54.10	53.40				
21. WEIGHT DRY SOIL *(18.-20.)*	gm	4.23	6.27	1.80	1.47				
22. WATER CONTENT $\left(\frac{19.}{21.} \times 100\right)$	PERCENT	44.0	44.2	22.2	22.4				
23. AVERAGE WATER CONTENT	PERCENT	44.1		22.3					

DRY UNIT WEIGHT DETERMINATION

	UNITS	COMPUTED							
RUN NUMBER		1	2						
24. DRY UNIT WEIGHT $\left(16. \times \frac{100}{100 + 23.}\right)$	LB./CU. FT.	87.0	95.9						

REMARKS

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*
L. Perry	L. Perry	B. Higgins

Figure 2–27. Data sheet, wet unit weight and water content determination (reverse of DD form 1214).

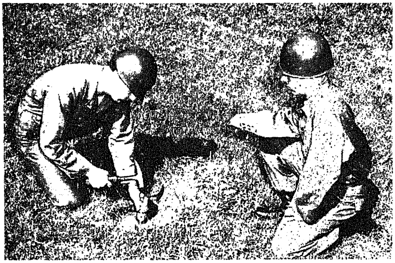


Figure 2–28. Driving sampler with hand-held hammer

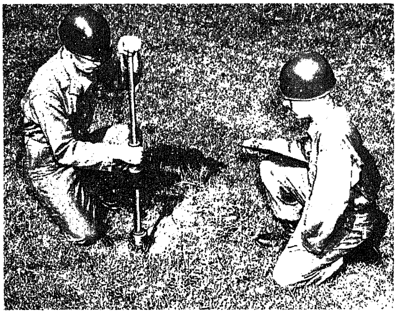


Figure 2–29. Driving sampler with drophammer.

2–18. Cylinder Samples by Soil Trafficability Sampler

a. The soil trafficability sampler (fig. 2–30) consists of a sample tube and an assembly to force the tube into the soil. It is forced by hand pressure, *not by blows from a hammer.* A movable piston is fitted within the cylinder and is attached to a rod which extends through the center of the drive tube and terminates in a flat dish or base plate at the upper end. The outer drive tube is attached to the sample cylinder at the bottom and has two handles at the top end. One of the handles is knurled and can be turned to lock the inner rod when the piston is in position. A long and a short spacer bar are bolted to the outer tube and are used to establish the size of the sample core. It is recommended that the sampler not be used for other than extremely soft and yielding soils. The walls of the cylinder are very thin and can be deformed if they come in contact with a hard object. Even hard or dry soil can damage the sampler. Its primary use is for samples to test the remolding characteristics in soils having initial low or very low supporting value.

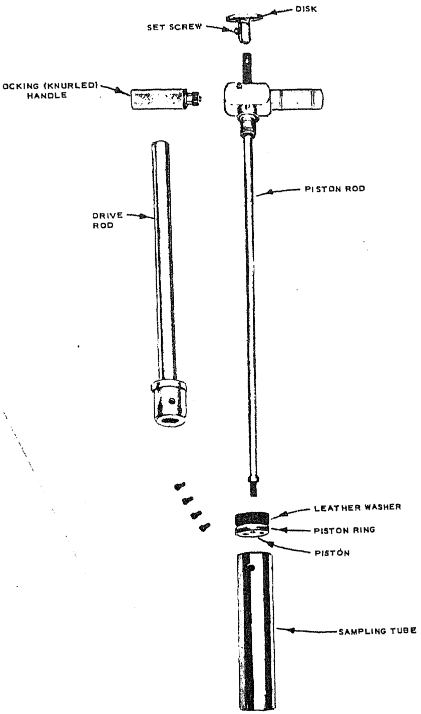


Figure 2–30. Soil trafficability sampler.

b. In operation, the piston is adjusted so that it is flush with the cutting edge of the sampler cylinder. The knurled handle is locked.

(1) The sampler is placed firmly in contact with the soil to be sampled.

(2) Holding the disk at the top to prevent vertical movement and unlocking the knurled handle, the sampler cylinder is forced into the soil.

(3) When the cylinder is fully in the soil, the knurled handle is locked to clamp the piston and keep the soil sample from slipping out due to the vacuum created.

(4) The entire sampler is now rotated a half turn to shear the soil at the base of the cylinder, then carefully withdrawn from the hole and inverted so that the dish becomes a base plate. There is a spud on the side of the sampler for releasing the side friction and the vacuum caused by withdrawing the sampler from the ground.

(5) The longer spacer bar is swung into position to act as a stop while the piston ejects the sample.

(6) The knurled handle is released and the drive tube is pushed until the spacer bar hits the base plate and a portion of the sample is pushed up out of the cylinder.

(7) The portion of the sample flush with the cutting edge of the sampler is cut off and discarded. This amount of soil discard offsets any uneven shearing at the bottom of the hole and gives the soil sample a true cylindrical shape.

(8) The short spacer bar is swung into position and the long bar is moved out of the way.

(9) The soil sample is ejected until the short bar stops the action. Some of the soil sample will still be in the cylinder.

(10) Using the sample cutter (piano wire), the soil sample is cut off flush with the sampler cutting edge into a preformed plate made to fit around the cutter. The sample is now exactly 1.87 inches in diameter and 3.453 inches long.

(11) The remaining soil in the sampler is discarded.

c. This sampler can be used in conjunction with a hand auger to obtain cores at depths up to 48 inches below the surface.

NOTE

> If the hole dug by the auger does not cave in, the soil may be too dry for the piston sampler.

The cores are sealed as described in paragraph 2-16*c*.

d. The soil trafficability sampler requires proper maintenance and adjustment to produce consistent results.

(1) It is essential to keep the inside of the sampling tube, the piston ring, and the leather washer reasonably clean. After 5 to 25 samplings, depending on the type of soil, the tube should be immersed, first in water and then in fuel oil, working the piston up and down five or six times in each liquid. After the excess of fuel oil has been wiped off, light machine oil should be squirted into the tube. If the instrument becomes stiff and hard to work, the tube should be removed, the piston disassembled and thoroughly cleaned, and the leather washer oiled. Care should be taken in removing the tube to prevent its slipping from the head suddenly and bending the piston rod. Tube walls and cutting edges are relatively soft and should be handled with care.

(2) The effective piston rod length should be adjusted to keep the face of the piston flush with the cutting edge of the tube when the piston rod handle (disk) is fully depressed. This is done by loosening the setscrew on the handle, screwing the handle up or down to the correct position, and retightening the setscrew.

2-19. Cylinder Samples by CBR Mold

a. In soft, fine grained soils, cylinder samples for undisturbed California bearing ratio (CBR) or density tests may be taken directly in the CBR compaction cylinder by using the sampling collar (cutter) as illustrated in figure 2-31. First, the surface of the ground is smoothed and the sampling collar and mold pressed into the soil with moderate pressure. A trench is excavated around the cylinder (fig. 2-32). The mold is again pressed firmly down over the soil, using the hand driver or loading bar if necessary (fig. 2-33). A loading bar may be improvised from any piece of timber of suitable size. The

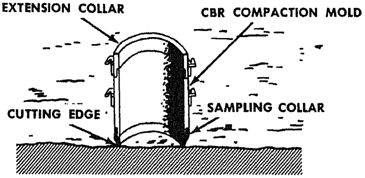


Figure 2-31. Section through CBR mold.

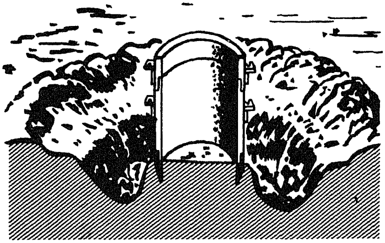


Figure 2-32. Trench cut around cylinder.

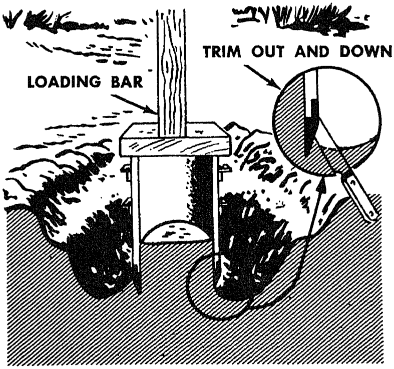


Figure 2-33. Using loading bar to drive cylinder.

soil is carefully trimmed away from the sampling collar with a knife, cutting downward and outward to avoid cutting into the sample. The actual cutting to size is done with the sampling collar. The sampler may be forced down with the field CBR jack, although this jack has only approximately 2 inches of travel. It is better to use a truck jack, if available. In either case, the sampler should not be forced down ahead of the trimming on the outside of the cylinder. The trench is then excavated deeper and the process repeated until the soil penetrates well into the extension collar (fig. 2–34). If stones interfere, they are picked out carefully and the space filled with soil. This fact is recorded in the log of the sample where pertinent.

b. The sample is cut off at the bottom of the mold, using a shovel, knife, or wire saw (fig. 2–35). The mold and sample are then removed from the hole.

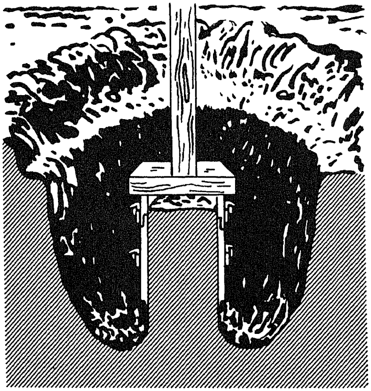


Figure 2–34. Cylinder in position prior to cutting sample.

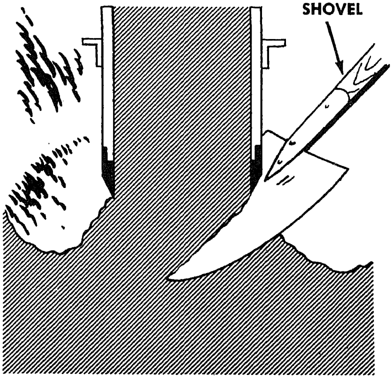


Figure 2–35. Cutting off cylindrical sample.

c. The upper collar is removed and the top surface of the sample trimmed down into the mold a distance of about ½ inch. This recess is filled with paraffin to seal the end of the sample (fig. 2–36). The mold is turned over and the cutting edge removed. This end is trimmed down into the mold about ½ inch, as before, and the resulting space filled with paraffin. If the sample is to be handled very much prior to testing, the ends should be overfilled with paraffin and then trimmed exactly flush with a straightedge. Boards are placed over each end and clamped in place by the use of bolts, wire, or string (fig. 2–37). Samples which must be transported some distance or which may have to be handled quite a bit before testing should be wrapped in cloth and soaked in paraffin in layers.

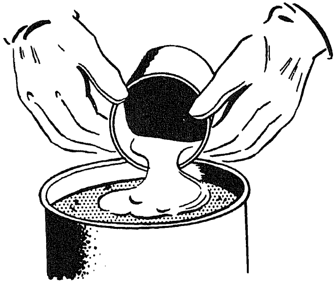


Figure 2–36. Sealing sample in CBR mold.

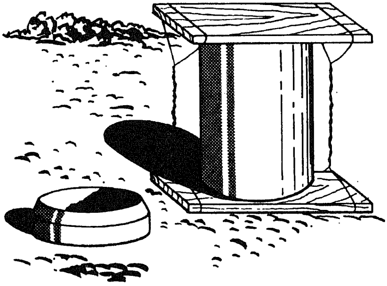


Figure 2–37. Protecting sample in CBR mold.

2–20. Cylinder Samples by Expedient Methods

Under expedient conditions, undisturbed samples may be taken by the use of a large cylindrical tin can from which the top and bottom have been removed. A short section of pipe may be similarly used, first sharpening one end to a cutting edge, the diameter of which is equal to the inside diameter of the pipe. The method of securing, trimming, and sealing the sample in such cases is similar to that described in paragraph 2–16.

2–21. Box Samples

Box samples sometimes are used for large undisturbed samples requiring extensive investigation. They can be packed firmly for shipment or storage, but require considerable paraffin. The process of obtaining a box sample is shown in figures 2–38, 2–39, and 2–40. As indicated, an excavation is made as for a chunk sample. The sample is trimmed to a size slightly smaller than the size of the box. The top and bottom are removed from the box, which is then placed over the sample (fig. 2–38). The space between the sides of the sample and the inside of the box is filled with melted paraffin, after which paraffin is poured over the top of the sample, and the top of the box replaced (fig. 2–39). This end is marked "TOP". The sample is cut off, and the box and sample removed from the hole (fig. 2–40). The bottom of the box is then replaced.

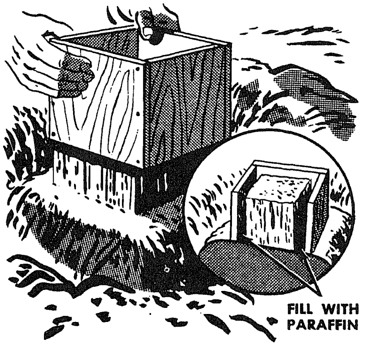


Figure 2–38. Securing a box sample.

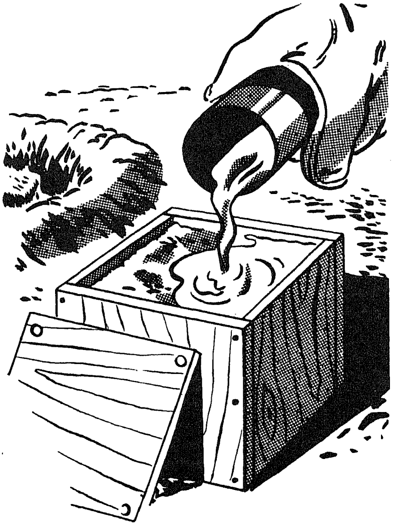


Figure 2–39. Sealing a box sample.

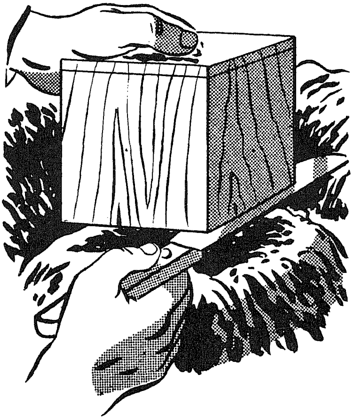


Figure 2–40. Cutting off a box sample.

2–22. Quartering Samples

The process of reducing a representative sample to a convenient size, or of dividing a sample into two or more smaller samples for testing, is called quartering. The procedure to be used varies somewhat, depending on the size of the sample.

a. Samples Weighing over 100 Pounds. Quartering of a sample of this size is shown in figures 2–41, 2–42, 2–43, and 2–44. The sample is first mixed and piled on the quartering canvas (fig. 2–41). Each shovelful

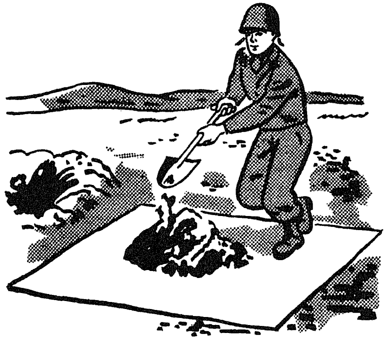


Figure 2–41. Mixing and piling a sample weighing more than 100 pounds.

is placed in the center of the cone so that the soil runs down evenly in all directions in order to mix the sample. The cone is then flattened with the shovel, spreading the material to a circular layer of approximately uniform thickness (fig. 2–42). A stick or pipe is inserted under the center of the pile (under the canvas) and both ends of the stick are lifted, thus dividing the sample into two parts (fig. 2–43). The stick is removed, leaving a fold in the canvas. The stick is again inserted under the pile, this time at right angles to the first division, and again lifted, dividing the sample in four parts (fig. 2–44). Two diagonally opposite quarters are discarded, being careful to clean the fines from the canvas. The remaining material is remixed by taking alternate shovelfuls from each quarter. The quartering process is repeated as necessary to reduce the sample to the desired size.

b. Samples Weighing 25 to 100 Pounds. In quartering this size of sample, the soil is piled on the canvas and mixed by alternately lifting the corners of the canvas and pulling over the samples as if preparing to fold the canvas diagonally, as illustrated in figure 2–45. The sample is then flattened and quartered, as in *a* above.

c. Samples Weighing Less Than 25 Pounds. The process of quartering samples less than 25 pounds is similar to the 100 pound and more process. The sample is placed on the canvas or a clean sheet of paper. It is mixed thoroughly with a trowel and formed into a conical pile (fig. 2–46). The cone is flattened by pressing downward with the trowel (fig. 2–47). The trowel is used to divide the sample

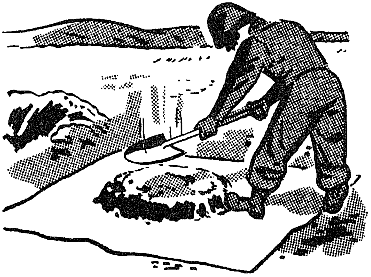


Figure 2–42. Flattening sample to uniform thickness.

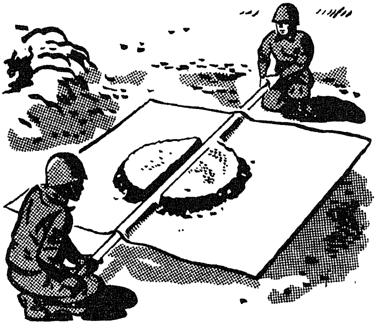


Figure 2–43. Halving the sample.

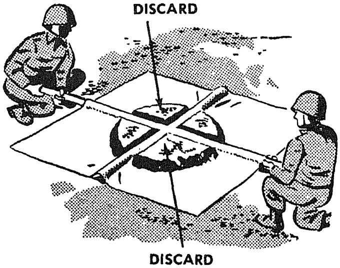


Figure 2–44. Quartering the sample.

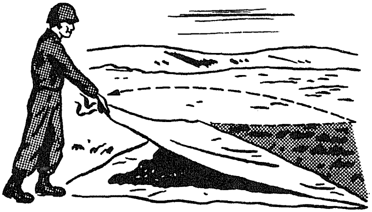


Figure 2–45. Mixing a sample weighing 25 to 100 pounds.

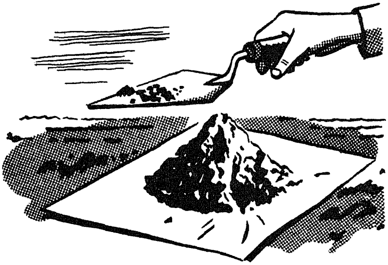


Figure 2–46. Preparing a sample weighing less than 2 pounds.

into quarters. Diagonally opposite quarters are discarded (fig. 2-48). The process is repeated as necessary to reduce the size of the sample for testing.

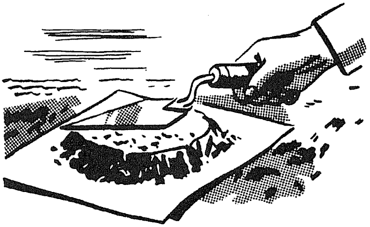


Figure 2-47. Flattening a small sample.

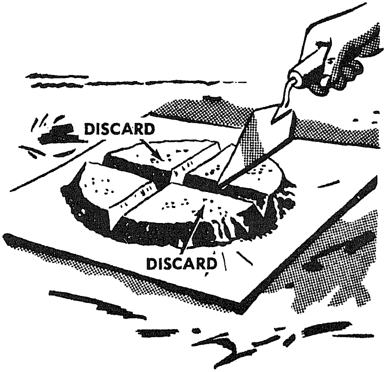


Figure 2-48. Quartering a small sample.

Section III. SIEVE ANALYSIS

2-23. Description

Soil grains and gradation are discussed in paragraph 2-2, with some consideration of the effects of grain characteristics on the physical properties of soils. The use of grain size and grain-size distribution in soil classification is explained in paragraph 2-3 and visual-manual tests and their use for field identification are covered in paragraphs 2-8 and 2-9. Although estimates of grain size of coarser materials may be made in this way, the accurate determination of the grain-size distribution or gradation of coarse soil fractions requires a sieve analysis, or grain-size analysis. The proportion of material of each grain size present in a given soil is determined by this test. Sieve separation of coarse and fine fractions of mixed soil is required to provide samples for further testing.

a. The term *grain-size distribution* refers to the portion of soil particles of different sizes which are contained in a given soil. The determination of the grain-size distribution of a soil, also called *mechanical analysis*, may be accomplished by a screening process (*sieve analysis*) for coarse-grained or cohesionless soils, by a settling process in water (*wet mechanical analysis* or *hydrometer analysis*) for fine-grained or cohesive soils, or by both the screening and settling processes on the same sample (*combined mechanical analysis*). Any grain-size distribution test is valid only when the soil particles are completely separated one from the other, so that the particles can pass through the sieve opening or settle through the water individually. Sieve analysis is 5 applicable to soil containing small amounts of material passing the No. 200 sieve provided the grain-size distribution of that portion passing the No. 200 sieve is not of interest. The sieve analysis is conducted either on the entire sample or on the sample after the fines are removed by prewashing.

b. The Corps of Engineers specifies a washed sieve analysis with prewashing unless otherwise allowed by the specifications. Even the most coarse grained and apparently the cleanest soil will at times produce a higher-than-expected percentage of fine particles. For military expediency, or when time is critical, prewashing may be eliminated. If there is any doubt as to the need for prewashing, a moist pat of the soil may be quickly dried on the stove or in the oven and its dry strength examined by crushing between the fingers. If it can be crushed and powdered easily, a sieve analysis can be made without prewashing (dry sieve analysis). In preparing a sample for dry sieve analysis, however, it may still be necessary to pulverize the finer portion of the sample to insure separation of the particles. The size of sample required for a sieve analysis depends upon its particle size as follows:

Maximum particle size (inches)	*Minimum weight of sample (grams)*
3	6000
2	4000
1	2000
½	1000
Finer than No. 4 sieve	200

c. A majority of the large samples must be dried in preparation for tests such as mechanical analysis, compaction, and CBR.

(1) *Air drying.* The soil is placed in pans or spread on a canvas and exposed to the direct rays of the sun, or to a draft of dry air. Frequent agitation or stirring speeds up the drying. A fan blow-

ing air over the sample will greatly increase the rate of drying. Care must be taken not to blow away the fine material.

(2) *Oven-drying.* Samples frequently are dried in an oven to constant weight. This is done most frequently in connection with the determination of the moisture (water) content of the soil.

(3) *Direct heat.* Some samples, particularly coarse grained soils which do not contain appreciable amounts of organic matter, can be dried expeditiously in a frying pan over a hotplate or gasoline burner, if available.

2–24. Equipment

The following equipment (fig. 2–49) is provided in the soil testing set for sieve analysis:

Balance, specific gravity determination, 2 gram capacity (not shown)
Brush, varnish, 1-inch
Cover, test sieve, 8-inch diameter
Cloth, cotton duck, 60 x 60 inches
Mortar, porcelain
Oven, laboratory (not shown)
Pan, test sieve, 8-inch diameter
Pans or evaporating dishes, as required
Pestle, porcelain-tipped
Scale, bench, 3 weigh beams, 2610-gram capa
Scale, bench, twin weigh beams, 21,100-gr capacity
Scoop, kitchen
Shaker, testing sieve, laboratory, hand-opera (fig. 2–50)

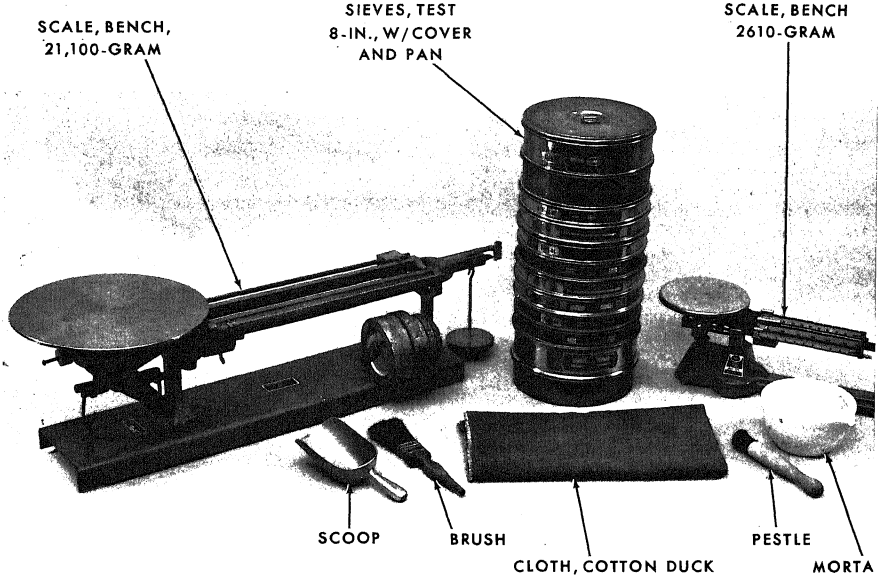


Figure 2–49. Equipment for sieve analysis.

Sieves, test, 8 inch diameter:
- Full Height (2 in. deep): 2, 1½, 1, ¾, and ¼ inch.
- Half Height (1 in. deep): U.S. No. 4, 10, 40, 60, 100 and 200* (fig. 2-51).

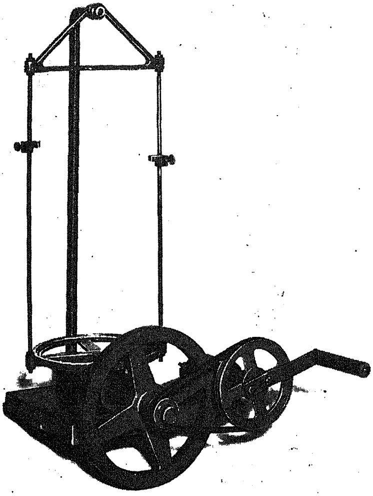


Figure 2-50. Hand operated sieve shaker.

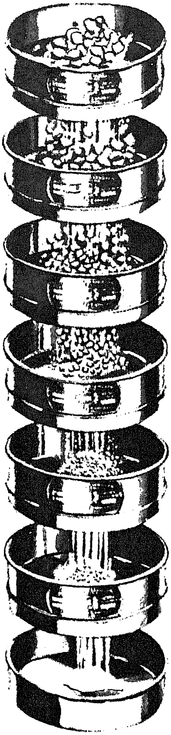


Figure 2-51. Testing sieves for dry sieve analysis.

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*Note. A full set of sieves includes 3 and ⅜ inch, but not ¼ inch and U.S. No. 20, No. 140 but not No. 100, in addition to the above. A set consisting of 3, 1½, ¾, and ⅜ inch, U.S. No. 4, 8, 16, 30, 50, 100 and 200 is recommended by A.S.T.M. to give uniform spacing of points on the grain-size distribution graph, since each opening is twice the size of the one below.

2–25. Dry Sieve Analysis

a. Preparation. Material to be analyzed using the dry sieve method is first air-dried, after which any clusters of soil are thoroughly broken up using the fingers or a mortar and pestile. Care must be taken only to separate the individual particles and not to crush any of them. If the sample is larger than the recommended amount, it should be reduced in size by quartering (para 2–22) until a representative sample of about the correct size (para 2-23*b*) is obtained. Too large a sample can overload a sieve and result in incomplete separation and errors in the test.

b. Procedure. The prepared sample is now ready for the actual test which is conducted according to the following directions:

(1) Record all identifying information of the sample, such as project, excavation number, sample number, description of sample, and date on a DD Form 1206 (Sieve Analysis Data Sheet) (fig. 2–52).

(2) Oven-dry the sample at 110° ± 5° C., and allow it to cool.

(3) Weigh the dried and cooled sample and record the weight as the weight of original sample.

(4) Select a nest of sieves suitable to the soil being tested. Weigh each selected sieve and record its weight. The top sieve, if possible, should be able to pass the largest particles in the sample. Particles larger than the largest sieve must be measured. The remaining sieves are selected based on the knowledge of the use to which the grain-size distribution curve will be put. Lacking this information, all the sieves below the largest one should be used. The sieves issued in the test set, in order of decreasing size openings, shown in parentheses (in millimeters), are as follows:

2-inch (50.8-mm)
1½-inch (38.1-mm)
1 inch (25.4-mm)
¾ inch (19.0-mm)
¼ inch (6.35-mm)
No. 4 (4.76-mm)
No. 10 (2.00-mm)
No. 40 (0.420-mm)
No. 60 (0.250-mm)
No. 100 (0.149-mm)
No. 200 (0.074-mm)

(5) Arrange the sieves in a nest with the largest opening sieve on top and each progressively smaller opening sieve in order below it. Attach the bottom pan to the smallest sieve used.

(6) Place the sample on the top sieve of the nest and put the cover plate on.

(7) Place the nest of sieves in the shaker and shake the nest for about 10 to 15 minutes. The length of time depends on the amount of fine material in the sample. Shaking should continue until the amount of material on any sieve does not show appreciable change.

(8) The nest may be shaken by hand if a shaker is not available. Use a lateral and vertical motion accompanied by a slight jarring to keep the sample moving continuously over the screens. Jarring is accomplished by dropping the nest lightly on a stack of papers or magazines. Use extreme caution not to "break" the nest and do not open it to rearrange or manipulate the particles.

(9) When the shaking is completed, set the nest down and remove the top cover.

(10) Start with the top sieve. Weigh it and the material retained on it.

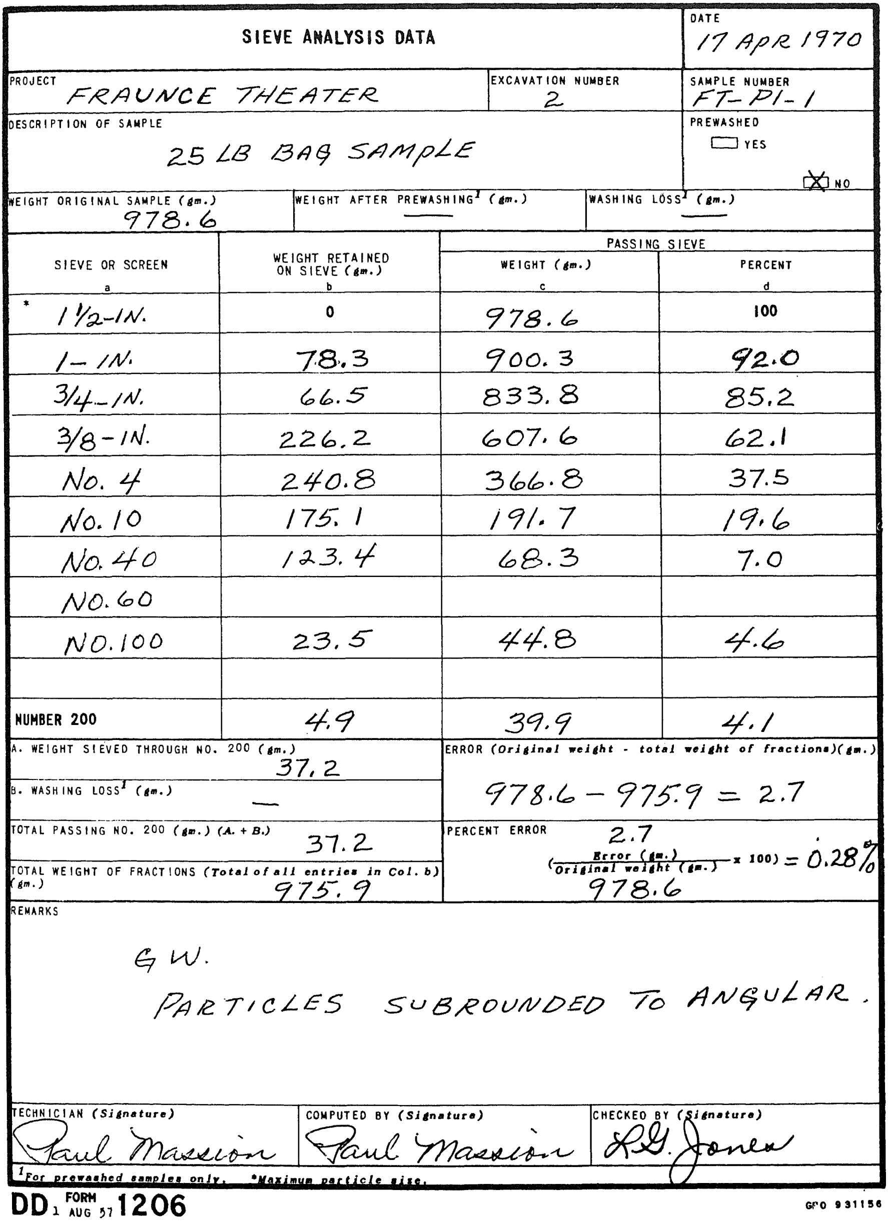
(11) Record this weight (sieve and sample) on the line opposite the sieve size.

(12) Set the weighed sieve and material aside until the weighing of all sieves is completed. *Do not discard* any of the material.

(13) Remove the next sieve and material, weigh, record, and set the sieve and material aside.

(14) Continue this process for each sieve *and the bottom pan* in the nest.

(15) Subtract the weight of the empty sieve or pan from the sieve plus material weight. Total the individual material weights and compare the total to the original sample weight, (3) above. If the difference exceeds one percent, check each sieve fraction for possible error. If no error has been made in the weighing, but the weight error still exceeds one percent, rerun the test.



SIEVE ANALYSIS DATA

DATE 17 APR 1970

PROJECT FRAUNCE THEATER

EXCAVATION NUMBER 2

SAMPLE NUMBER FT-P1-1

DESCRIPTION OF SAMPLE 25 LB BAG SAMPLE

PREWASHED ☐ YES ☒ NO

WEIGHT ORIGINAL SAMPLE (gm.) 978.6

WEIGHT AFTER PREWASHING[1] (gm.) —

WASHING LOSS[1] (gm.) —

SIEVE OR SCREEN a	WEIGHT RETAINED ON SIEVE (gm.) b	PASSING SIEVE WEIGHT (gm.) c	PASSING SIEVE PERCENT d
* 1½-IN.	0	978.6	100
1-IN.	78.3	900.3	92.0
3/4-IN.	66.5	833.8	85.2
3/8-IN.	226.2	607.6	62.1
No. 4	240.8	366.8	37.5
No. 10	175.1	191.7	19.6
No. 40	123.4	68.3	7.0
NO. 60			
NO. 100	23.5	44.8	4.6
NUMBER 200	4.9	39.9	4.1

A. WEIGHT SIEVED THROUGH NO. 200 (gm.) 37.2

B. WASHING LOSS[1] (gm.) —

TOTAL PASSING NO. 200 (gm.) (A. + B.) 37.2

TOTAL WEIGHT OF FRACTIONS (Total of all entries in Col. b) (gm.) 975.9

ERROR (Original weight - total weight of fractions)(gm.) 978.6 − 975.9 = 2.7

PERCENT ERROR $\left(\frac{\text{Error (gm.)}}{\text{Original weight (gm.)}} \times 100\right) = 0.28\%$

2.7
978.6

REMARKS

GW.

PARTICLES SUBROUNDED TO ANGULAR.

TECHNICIAN (Signature) Paul Massion

COMPUTED BY (Signature) Paul Massion

CHECKED BY (Signature) L.G. Jones

[1] For prewashed samples only. *Maximum particle size.

DD FORM 1 AUG 57 1206

GPO 931156

Figure 2-52. Data sheet, example of dry sieve analysis.

2–26. Sieve Analysis With Prewashing

Samples containing cohesive soil must be washed prior to sieve analysis to break up the lumps and to remove the coating of cohesive particles.

a. Preparation. The sample is oven-dried and weighed, then it is placed in a clean container and clean water added to cover the sample completely. Allow the sample to soak until the adhering and lumpy particles are completely disintegrated. This process may take from 2 to 24 hours depending on the amount and plasticity of the fine material. Stirring and breaking up lumps will hasten the process.

b. Procedure. When the sample has disintegrated and no lumps exist, it is ready for prewashing according to the following procedures:

(1) Transfer the sample and water from the container to a No. 200 sieve. If the sample contains an appreciable amount of coarse particles, use a nest of a No. 4 or a No. 40 sieve and a No. 200 sieve to prevent overloading the smaller opening sieve. Large particles may be washed individually over the No. 200 sieve (no more than 200 grams at a time) and set aside for future use.

(2) Wash the sample thoroughly until the material on the sieve and the wash water appear to be clean (fig. 2–53). Rub or stir the sample carefully using the fingers.

(3) The wash water and the material which has passed through the No. 200 sieve are discarded (when only dry sieve analysis is required).

(4) The washed sample and any separately washed large particles are oven dried and reweighed. The difference in weight between the original oven-dried sample and this weight sample is entered as a washing loss, included in the pan total (fig. 2–52).

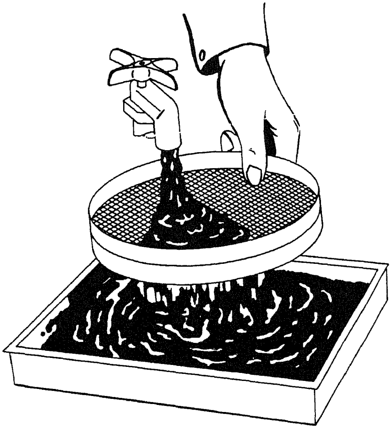


Figure 2–53. Prewashing a sample.

(5) The washed and dried sample is dry-sieve tested as described in paragraph 2–25*b*(4) through (15).

(6) In recording the weights, the amount passing the No. 200 sieve (weight retained in pan) is added to the washing loss to obtain the pan total weight. This pan total is added to the other fraction weights to compute the total weight after testing and the error in weights.

2–27. Calculations and Curve Plotting

The figures as recorded during the test are used to compute the percentages and then to compile a grain-size distribution curve.

a. Computations.

(1) The weight retained on each sieve times 100 is divided by the total weight to give the partial percent retained:

$$\frac{\text{weight retained on sieve}}{\text{total sample weight}} \times 100 = \text{partial percent retained.}$$

(2) Add the percent retained for a particular sieve to the total of all the partial percent retained values for the larger sieves to give the total percent retained. The percent finer than any sieve size is found by subtracting the total percent retained from 100.

Example: weight retained on No. 10 sieve = 175.1 g.

$$\text{partial percent retained} = \frac{175.1}{978.6} \times 100 = 17.9\%$$

$$\text{total percent retained} = 8.0 + 6.8 + 23.1 + 24.6 + 17.9 = 80.4$$

$$\text{percent fines by weight} = 100 - 80.4 = 19.6\%$$

This value is recorded in the last column of figure 2–52.

(3) The total of the weights retained on the sieves plus the total passing the No. 200 sieve (pan total) are compared to the original weight. This error (2.7 g. in fig. 2–52) is within the allowable limits.

b. Plotting. DD Form 1207 (Grain-size Distribution Graph) (fig. 2–54) is used to show the sieve analysis results in a graphic form. The graph uses semilogarithmic coordinates. The grain sizes are plotted on a logarithmic scale while the percentages are plotted on an arithmetical scale. The grain sizes may be plotted using the sieve number along the top edge or grain diameters (in millimeters) along the bottom edges. The percentage values from DD Form 1206 are plotted and a smooth curve is drawn connecting the plotted points. The curve for the sample FT–PI–I (fig. 2–52) is shown as one of the curves on figure 2–54. This is the final product of a sieve analyisis.

Significance of Grain-size Distribution Curve.
s curve affords ready visualization of the distri-
ion and range of particle sizes, and is particularly
›ful in determining the soil classification and usa-
ty of the soil as a foundation or construction
erial. Coefficients of uniformity and curvature,
l in the Unified Soil Classification System to
sify sands and gravels as to their gradation
racteristics, are determined from the grain-size
ribution curve as follows:

(1) The grain-size, in millimeters, which cor-
onds to 10 percent passing on the grain-size dis-
ution curve is called *Hazen's effective size;* it is
gnated by the symbol D_{10}. The *uniformity co-*
ient (C_u) is the ratio between the grain diameter,
nillimeters, corresponding to 60 percent passing
he curve (that is D_{60}) and the 10 percent passing
o). Hence, $C_u=D_{60}/D_{10}$.

(2) The *coefficient of curvature*, designated by
symbol C_c, is written $C_c=(D_{30})^2/(D_{60}\times D_{10})$.
and D_{60} have meanings previously assigned while D_{30} is the grain diameter, in millimeters, corresponding to 30 percent passing on the grain size distribution curve.

(3) For example, the uniformity coefficient and coefficient of curvature of the well-graded sand, SW, in figure 2-54 (DD Form 1207 (Grain Size Distribution Graph-Aggregate Grading Chart)) is determined from the grain-sized distribution curve as follows:

(a) D_{60} reads 0.72 mm

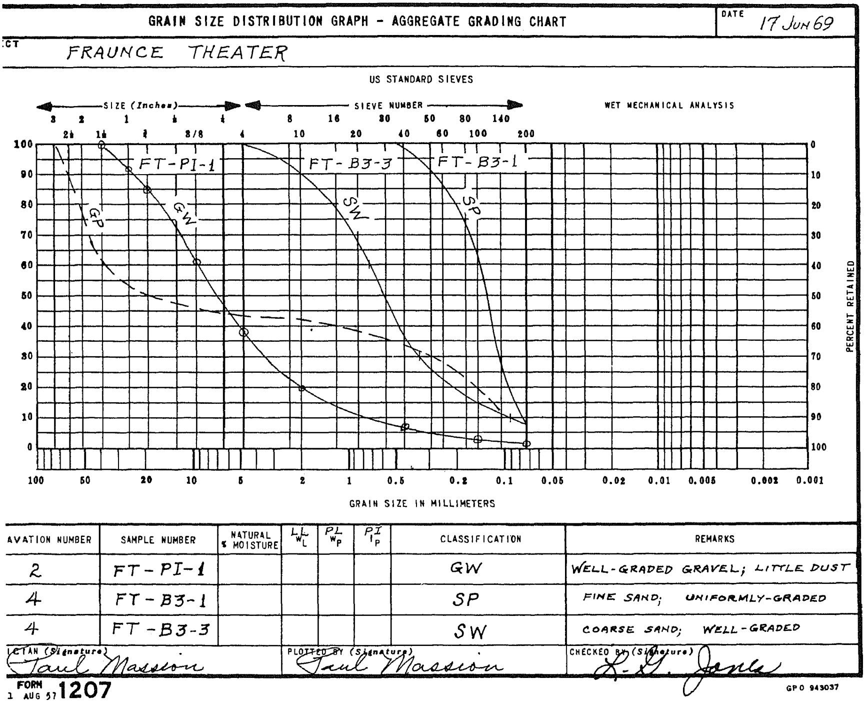
(b) D_{30} reads 0.36 mm

(c) D_{10} reads 0.09 mm

(d) $C_u=\dfrac{0.72}{0.09}=8.0$

(e) $C_c=\dfrac{0.36\times0.36}{0.72\times0.09}=2.0$

(f) Both C_u and C_c meet criteria shown in table B-1 (Appendix B) for well-graded sand, SW (C_u greater than 6 and C_c between 1 and 3).



GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 17 Jun 69

:CT FRAUNCE THEATER

AVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	LL W_L	PL W_P	PI I_P	CLASSIFICATION	REMARKS
2	FT-PI-1					GW	WELL-GRADED GRAVEL; LITTLE DUST
4	FT-B3-1					SP	FINE SAND; UNIFORMLY-GRADED
4	FT-B3-3					SW	COARSE SAND; WELL-GRADED

ETAN (Signature) Paul Massion

PLOTTED BY (Signature) Paul Massion

CHECKED BY (Signature) L. G. Jones

FORM 1 AUG 57 1207

GPO 943037

Figure 2-54. Grain-size distribution curves from sieve analysis.

Section IV. SPECIFIC GRAVITY

2–28. Introduction

A general definition for specific gravity of a solid substance is the ratio of its weight to the weight of an equal volume of water. For exact analysis, the specifications require distilled water, and all measurements of water and solids to be made at stated temperatures. In dealing with soils, the specific gravity is necessary for certain soil tests, such as the wet mechanical analysis. It is also necessary for computations involving volume or weight relationships, such as computing the void ratio, percent saturation, and similar values. Specific gravity is not too useful as an aid to soil identification since it varies over a comparatively narrow range for most soils. Specific gravity of a soil is largely dependent upon the minerals from which it was derived and their specific gravity. However, as a general guide, some typical values can be given for soils based on their organic content.

a. For most inorganic soils, specific gravity ranges from 2.60 to 2.80. Many within this group fall between 2.65 and 2.75. Occasional soils derived from unusually heavy material may have a specific gravity as high as 3.00.

b. Soils with large amounts of organic matter or porous particles (such as diatomaceous earths) have specific gravities below 2.60. Some range as low as 2.00.

2–29. Specific Gravity of Soil

The specific gravity of a soil mass can be expressed in one of three different forms: the specific gravity of the solids, the apparent specific gravity, and the bulk specific gravity.

a. The *specific gravity of solids* is the ratio of the weight in air of a given volume of soil solids to the weight of an equal volume of distilled water, both at a stated temperature.

b. The *apparent specific gravity* of a soil is the ratio of the weight in air of a given volume of the impermeable portion to the weight in air of an equal volume of distilled water at a stated temperature. The impermeable portion of a porous or permeable material such as soil grains includes the solid material plus the impermeable pores or voids within the particles.

c. *Bulk specific gravity* is the ratio of the weight in air of a given volume of a permeable material (including permeable and impermeable void spaces) to the weight of an equal volume of distilled water at a stated temperature.

d. Soil particles generally contain pore space or voids that may be large enough to be visible or may be of such minute size as to be undetectable even at high magnification. Specific gravity of solids thus can be measured only when particles are of such small size that all voids will be filled with the water used to measure the volumes. The test procedure must ensure that no air entrapped in pores is included in the measured particle volume. The test procedure for *apparent specific gravity* requires the application of a partial vacuum or boiling to remove air from pores. Then water can enter (permeate) these pores and such permeable pores are not measured as part of the given volume. However, most soil particles, even as small as sand grains, will contain pores that are impermeable under these conditions, hence are included in the measured volume, and cause the *apparent specific gravity* to be less than the *specific gravity of solids*. In measuring the sample volume for determination of *bulk specific gravity*, all pores that will hold capillary water are included since the particles are weighed in a saturated condition.

2–30. Apparatus and Sample Selection

a. *Apparatus.* Some of the apparatus used to perform specific gravity tests (figure 2–55) includes:

Balance, 200-gram capacity (not shown)
Boxes, moisture
Dishes, evaporating
Filler, battery
Flask, volumetric, 500-ml capacity (not in test set)
Funnel, metal (not shown)
Hotplate, electric
Mortar and pestle
Pump, vacuum (not shown)
Scale, bench, 3 weigh beams, 2610-gram capacity
Stirrer, soil dispersion (not in test set)
Thermometer, general laboratory

The test also requires some equipment that is not in the test set but must be procured from commercial sources. Otherwise, use expedient substitutes from local sources.

b. *Sample Selection.* Particular care must be taken to obtain representative samples for a specific gravity test.

(1) It is ususally easier to begin the test with an oven-dried sample. However, some soils, particularly those with a high organic content, are difficult to rewet. These should be tested at their natural water content first, if possible, and the oven-dried weight determined at the end of the test.

(2) When the sample contains large and small particles, it should be separated on the No. 4 sieve. The specific gravity of the fine fraction and the apparent specific gravity of the coarse fraction are determined separately. A composite specific gravity for the entire sample is computed on the basis of the percent, by weight, of the fine and coarse material.

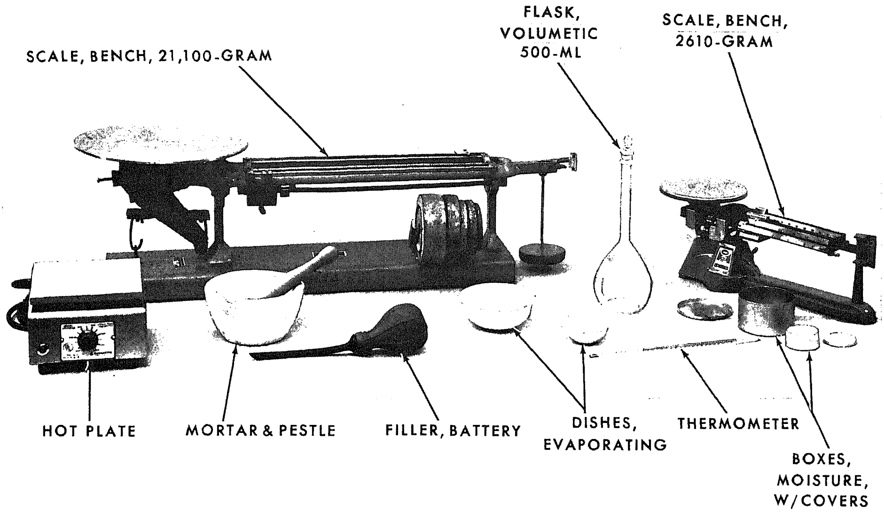


Figure 2-55. Apparatus for determining specific gravity of soils.

(3) If the specific gravity is to be used in conjunction with a hydrometer analysis (section V), it must be determined only on the fraction which is required for hydrometer analysis. This fraction is defined as that which passes through the No. 200 sieve. Note that the hydrometer is not included in the test set.

2-31. Procedures

The specific gravity test is made in one of two ways depending on whether the requirements call for soil at natural water content or oven-dried soil. The weight of the volumetric flask and water must be calibrated over the range of temperatures likely to be encountered in the laboratory. Selection of a representative soil sample and its handling during transfers from container to container are very critical in this testing procedure.

2-32. Calibration of the Volumetric Flask

Calibration consists of direct weighing of the flash and water over a selected temperature range.

a. Determine the dry weight of the flask (W_b).

b. Fill the flask with distilled water to slightly below the calibration mark.

c. Place the filled flask into a water bath of about 30 to 35 degrees centigrade and allow it to remain until the water and flask reach the bath temperature.

d. Remove the flask from the bath and add or remove a slight amount of water until the bottom of the meniscus is even with the calibration mark. Carefully dry the outside of the flask and remove any excess moisture which may be adhering to the inside of the neck above water surface.

e. Weigh the flask and water to the nearest 0.01 g.

f. Immediately after the weighing, shake the flask gently to mix the water and immerse a thermometer to about mid-depth in the water. Read and estimate the temperature of the water to the nearest 0.1° centigrade.

g. Repeat the procedures of *b* to *f* above, at approximately the same temperature (30 to 35° C.).

h. Make two more determinations, *b* to *g* above; one at room temperature and another at approximately 5° C cooler than room temperature.

i. Plot the weight of flask and water (W_{bw}) against the temperature (T) for each determination and draw a smooth curve through the plotted points (fig. 2-56). This is the calibration curve for the particular flask.

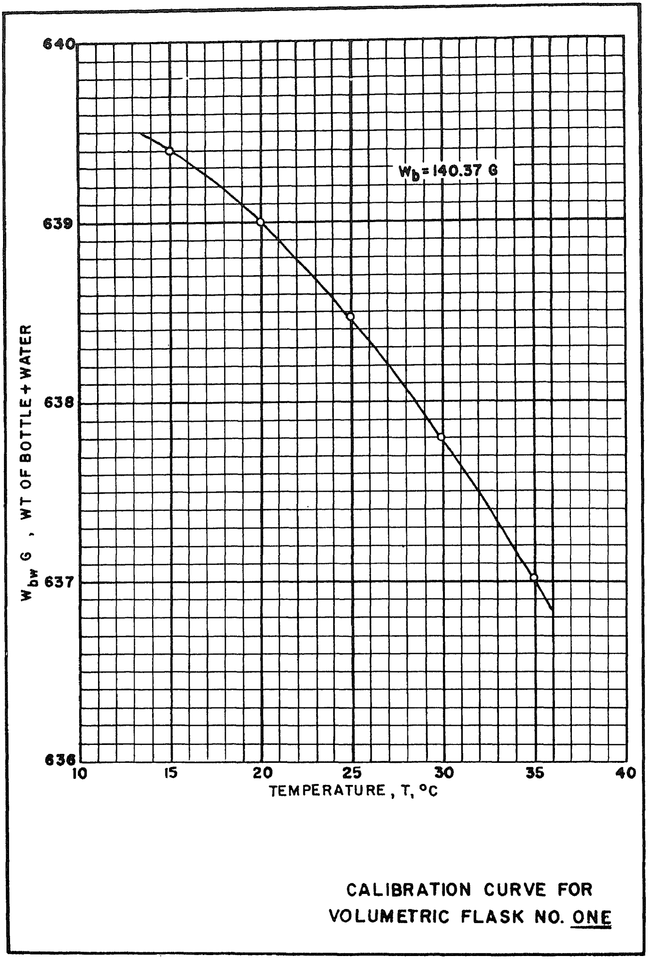


Figure 2–56. Calibration curve for volumetric flasks.

2–33. Testing

Specific gravity tests are usually easier to make if they begin with an oven-dried sample. However, they may begin with the natural water content still present in the soil, and the over-dried weights obtained after the test has been performed.

a. Soils With Natural Water Content.

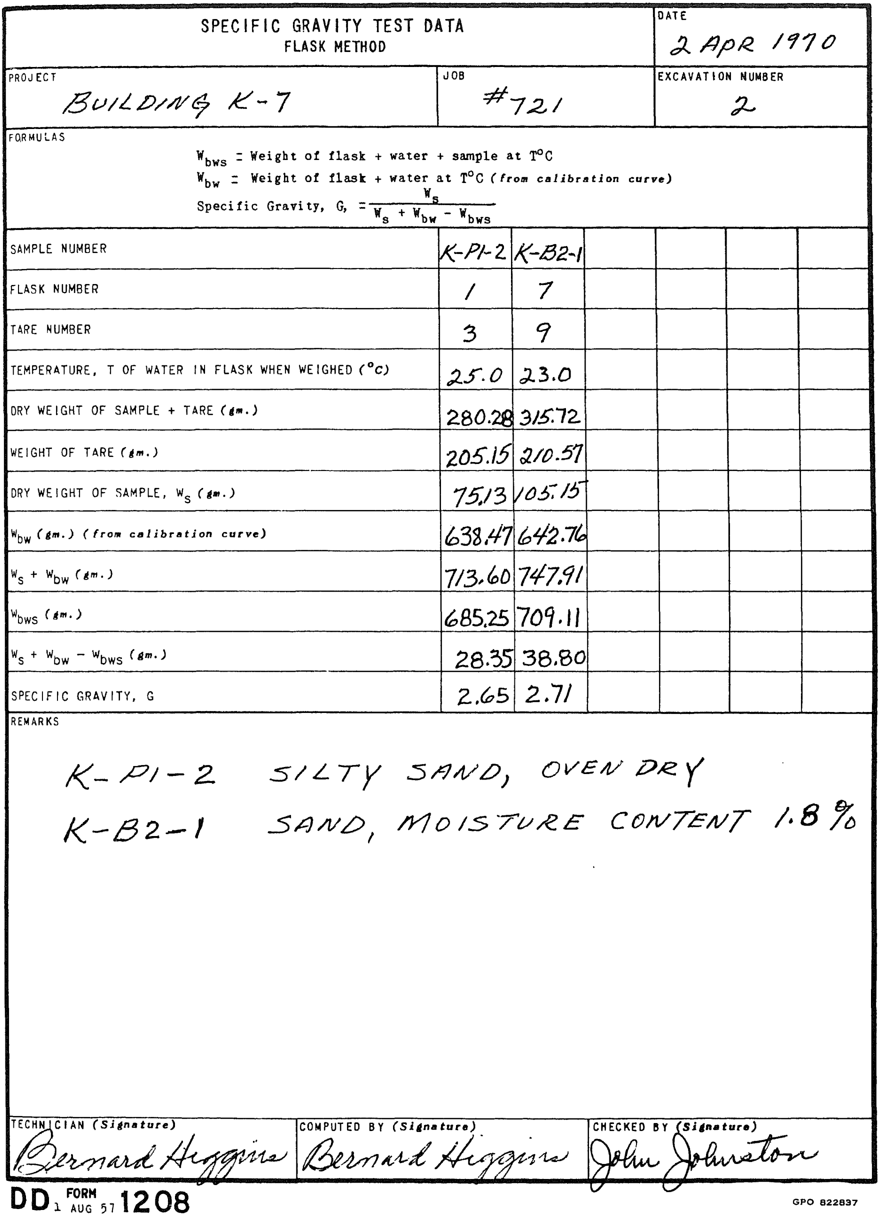
(1) Record all identifying information for the sample (DD form 1208 (Specific Gravity Test Data-Flask Method), fig. 2–57).

(2) Select a representative sample of approximately 50 to 80 g and place it in a dish.

(3) Add sufficient distilled water to form a slurry while mixing with a spatula.

(4) Transfer the slurry to the volumetric flask, and add distilled water until the flask is half full. Allow the suspension to soak 4 to 6 hours. Soaking is not required on clean sands.

(5) Connect the flask to a vacuum line and apply a vacuum of approximately 29.5 inches of mercury. Agitate the flask gently at intervals during the evac-



SPECIFIC GRAVITY TEST DATA
FLASK METHOD

DATE 2 APR 1970

PROJECT BUILDING K-7

JOB #721

EXCAVATION NUMBER 2

FORMULAS

W_{bws} = Weight of flask + water + sample at T°C

W_{bw} = Weight of flask + water at T°C *(from calibration curve)*

$$\text{Specific Gravity, } G, = \frac{W_s}{W_s + W_{bw} - W_{bws}}$$

SAMPLE NUMBER	K-P1-2	K-B2-1				
FLASK NUMBER	1	7				
TARE NUMBER	3	9				
TEMPERATURE, T OF WATER IN FLASK WHEN WEIGHED (°C)	25.0	23.0				
DRY WEIGHT OF SAMPLE + TARE *(gm.)*	280.28	315.72				
WEIGHT OF TARE *(gm.)*	205.15	210.57				
DRY WEIGHT OF SAMPLE, W_s *(gm.)*	75.13	105.15				
W_{bw} *(gm.) (from calibration curve)*	638.47	642.76				
$W_s + W_{bw}$ *(gm.)*	713.60	747.91				
W_{bws} *(gm.)*	685.25	709.11				
$W_s + W_{bw} - W_{bws}$ *(gm.)*	28.35	38.80				
SPECIFIC GRAVITY, G	2.65	2.71				

REMARKS

K-P1-2 SILTY SAND, OVEN DRY

K-B2-1 SAND, MOISTURE CONTENT 1.8%

TECHNICIAN *(Signature)* Bernard Higgins	COMPUTED BY *(Signature)* Bernard Higgins	CHECKED BY *(Signature)* John Johnston

DD FORM 1 AUG 57 1208

GPO 822837

Figure 2-57. Data sheet, specific gravity test for soils.

uation process. Soils of high plasticity will require about 6 to 8 hours to remove the air; soils of low plasticity about 4 to 6 hours. A vacuum pump is not available in the test set.

(6) Alternatively, entrapped air may be removed by boiling (slow rolling boil) the suspension gently for at least 10 minutes while occassionally "rolling" the flask to aid in the removal. Fast boiling may cause losses in the material being tested and should be avoided. Allow the flask and contents to cool before proceeding.

(7) Fill the flask with de-aired, distilled water to about ¾ inch below the 500-ml graduation and again boil to de-air the suspension.

(8) To test the de-airing, observe the water surface in the flask and carefully remove the stopper. The suspension can be considered de-aired sufficiently when the water surface in the flask is lowered less than ⅛ inch during this process.

(9) Using a pipette, add more water to the flask until the bottom of the meniscus is coincident with the calibration line on the neck of the flask.

(10) Thoroughly dry the outside of the flask and any moisture above the water surface inside the flask.

(11) Weigh the flask and contents to the nearest 0.01 gram and record the information. This is (W_{bws}).

(12) Immediately after weighing, stir the suspension to assure even temperature distribution and immerse a thermometer to the middepth of the flask. Read and record the temperature to the nearest 0.01° C.

(13) Carefully transfer the contents of the flask to an evaporating dish. Rinse the flask with distilled water until the sample is entirely transferred to the evaporating dish.

(14) Oven-dry the sample at a temperature of 100 ± 5° C. until it reaches a constant weight. Allow the soil to cool overnight in a desiccator to room temperature.

(15) Determine the dry weight of the sample, (W_s), to the nearest 0.01 g and record the weight on the data sheet.

b. Oven-Dried Soils. When the soil can be dried before the test, the procedure is as follows:

(1) Record the information identifying the soil sample.

(2) Oven-dry a representative fraction of approximately 100 g of the sample to a constant weight at 110±5° C., and cool in the desiccator to room temperature.

(3) Weigh the sample to the nearest 0.01 g and record the weight.

(4) Transfer the soil to a volumetric flask taking care not to lose or leave any material during the operation. As a check, reweigh the soil and flask after the transfer.

(5) Fill the flask about half full with de-aired, distilled water.

(6) Connect the flask to a vacuum line and apply a vacuum of approximately 29.5 inches of mercury for about 2 to 4 hours. Entrapped air may be removed as described in *a*(5) above, using the same precautions.

(7) Continue with the remainder of the test as described in *a*(6) to (12) above. Procedures *a*(13) to (15) can be used as a check on the initial weights.

2–34. Computations

a. The test furnishes the following information (first column in fig. 2–57):

(1) Weight of flask plus water plus solids at the test temperature (W_{bws} in grams).

(2) Weight of tare (container) plus dry soil (in grams).

(3) Weight of dry soil, (W_s), obtained by subtracting the weight of the container (tare) from (2) above.

(4) Weight of flask plus water at test temperature (W_{bw} in grams) obtained from the calibration curve (fig. 2–56).

b. To compute the specic gravity, (G_s), the following formula is used:

$$G_s = \frac{W_s K}{W_s + W_{bw} - W_{bws}}$$

where K is a correction factor based on the density of water at 20° C. (table 2–1).

Table 2-1. Relative Density of Water and Correction Factor (K) at Various Temperatures

Temp °C.	Relative density	Correction factor, K	Temp °C.	Relative density	Correction factor, K	Temp °C.	Relative density	Correction factor, K
18.0	0.99862	1.0004	23.0	0.99756	0.9993	28.0	0.99626	0.9980
.1	60	4	.1	54	3	.1	23	0
.2	58	4	.2	51	3	.2	20	0
.3	56	3	.3	49	3	.3	17	0
.4	54	3	.4	46	2	.4	14	0.9979
.5	52	3	.5	44	2	.5	11	9
.6	50	3	.6	42	2	.6	08	9
.7	49	3	.7	39	2	.7	06	8
.8	47	2	.8	37	1	.8	03	8
.9	45	2	.9	34	1	.9	00	8
19.0	0.99843	1.0002	24.0	0.99732	0.9991	29.0	0.99597	0.9977
.1	41	2	.1	29	1	.1	94	7
.2	39	2	.2	27	0	.2	91	7
.3	37	1	.3	24	0	.3	88	6
.4	35	1	.4	22	0	.4	85	6
.5	33	1	.5	20	0	.5	82	6
.6	31	1	.6	17	0.9989	.6	79	6
.7	29	1	.7	14	9	.7	76	5
.8	27	0	.8	12	9	.8	73	5
.9	25	0	.9	09	9	.9	70	5
20.0	0.99823	1.0000	25.0	0.99707	0.9988	30.0	0.99567	0.9974
.1	21	0	.1	04	8	.1	64	4
.2	19	0	.2	02	8	.2	61	4
.3	17	0.9999	.3	0.99699	8	.3	58	3
.4	15	9	.4	97	7	.4	55	3
.5	13	9	.5	94	7	.5	52	3
.6	10	9	.6	91	7	.6	49	3
.7	08	8	.7	89	7	.7	46	2
.8	06	8	.8	87	6	.8	43	2
.9	04	8	.9	84	6	.9	40	2
21.0	0.99802	0.9998	26.0	0.99681	0.9986	31.0	0.99537	0.9971
.1	00	8	.1	78	6	.1	33	1
.2	0.99798	8	.2	76	5	.2	30	1
.3	96	7	.3	73	5	.3	27	0
.4	93	7	.4	70	5	.4	24	0
.5	91	7	.5	68	4	.5	21	0
.6	89	7	.6	65	4	.6	18	0.9969
.7	87	6	.7	63	4	.7	15	9
.8	85	6	.8	60	4	.8	12	9
.9	83	6	.9	57	3	.9	09	9
22.0	0.99780	0.9996	27.0	0.99654	0.9983	32.0	0.99505	0.9968
.1	78	6	.1	51	3	.1	02	8
.2	75	5	.2	48	2	.2	0.99499	8
.3	73	5	.3	46	2	.3	96	7
.4	70	5	.4	43	2	.4	93	7
.5	68	5	.5	40	2	.5	90	7
.6	65	4	.6	37	1	.6	86	6
.7	63	4	.7	34	1	.7	83	6
.8	61	4	.8	32	1	.8	80	6
.9	58	4	.9	29	1	.9	77	5

Note. Relative density of water based on density of water at 4°C. equal to unity. Data obtained from Smithsonian Tables, compiled by various authors. Correction factor, K, is found by dividing the relative density of water at the test temperature by the relative density of water at 20°C.

2–35. Apparent and Bulk Specific Gravity

Apparent specific gravity is defined in paragraph 2–29*b*. Bulk specific gravity is defined in paragraph 2–29*c*. The test is described below.

a. Apparatus. The apparatus must be set up in a special manner to perform this test. The apparatus required for a bulk specific gravity test consists of the following:

(1) Balance (capacity of 5 kg or more; sensitive to 0.1 g).

(2) Mesh wire basket (approximately 8 inches in diameter and 8 inches high).

(3) Container (large enough to permit immersing the wire basket).

(4) Suitable equipment for suspending the wire basket from the center of the balance scale pan.

(5) Thermometer (range 0 to 50° C., graduated to 0.1°).

b. Sample Preparation. Bulk specific gravity is usually determined for the coarser materials (retained on the No. 4 sieve) of a soil sample. Large stones may be determined individually.

(1) The soil sample should be separated on a No. 4 sieve and the material retained on the sieve used for the test. A representative sample of approximately 5 kg is required.

(2) The sample should not be oven-dried if at all possible. Air-drying is permissible.

(3) The sample should be washed to remove dust or coatings from the surface and immersed in water for 24 hours.

(4) Just before making the test, the sample should be removed from the water and rolled in a large absorbent cloth to remove the excess surface moisture. Large particles may be wiped individually. The surfaces may still appear damp.

(5) Care should be taken to avoid excessive evaporation during the surface drying.

c. Test Procedures. The first two steps in the test should be performed as quickly as possible after the surface drying to avoid excessive evaporation.

(1) Weigh the saturated surface-dry sample and record the value to the nearest 1.0 g.

(2) Place the sample into the wire mesh basket and immerse the basket and sample into the water container.

(3) Hang the basket from the balance and support the container so that the basket hangs freely in the water.

(4) Determine the weight of the sample in water and record the value.

NOTE

The weight of the wire basket in water must be determined and substracted from the overall weight.

(5) Measure and record the water temperature.

(6) Oven-dry the sample at 110° ± 5° C., to a constant weight.

(7) After cooling the sample to room temperature, weigh the sample and record the weight.

d. Calculations. The test furnishes the dry soil weight (oven-dried), the saturated surface dry weight, and the weight of the sample in water. Table 2–1 lists the bulk correction factor (K) based on water temperature.

(1) To determine the bulk specific gravity (G_m):

$$G_m = \frac{\text{weight of dry soil in air} \times K}{\text{weight of saturated sample in air—weight of sample in water}}$$

(2) To determine the apparent specific gravity (G_a):

$$G_a = \frac{\text{weight of dry soil in air} \times K}{\text{weight of dry soil in air—weight of sample in water}}$$

(3) To determine the specific gravity (G) of an entire soil sample (larger and smaller than the No. 4 sieve) from the component determinations:

$$G = \frac{100}{\dfrac{\%\text{ passing No. 4 sieve}}{G_s} + \dfrac{\%\text{ retained on No. 4 sieve}}{G_a}}$$

e. Expedient Methods. The methods described above are the laboratory tests performed to determine the specific gravity of a soil sample. On occasion, the accuracy of these methods is not required or a quick determination is requested. The following factors should be considered:

(1) For instance, the water density correction factor (K) and the associated water temperature measurement (*c*(5) above) may be ignored in bulk specific gravity determination (*d* above).

(2) At times, there are stones present in the soil sample. An expedient method to determine the specific gravity of these particles is to tie a string or thread around the circumference and suspend the particle from the hook of a beam balance and weigh it to the nearest 0.1 gram (fig. 2–58). Note that this limits the minimum size of particle that can be tested by this method since it must be large enough to be tied with a string. The particle may come from an oven-dried or saturated sample. A can, glass beaker, or container of water is set on a platform below the balance and the particle is immersed in the water (fig. 2–59). The suspended particle is not permitted to rest on the bottom nor to touch the sides of the container. The sample is weighed again to the nearest 0.1 g. The weight of the thread or string is considered negligible and is disregarded in the calculations. The specific gravity (G_a) is calculated as follows:

$$G_a = \frac{\text{weight of sample in air}}{\text{weight of sample in air—weight of sample in water}}$$

(3) A simple, quick method uses only a graduated cylinder. The air-dried sample is weighed to the nearest 0.1 g. The graduate is filled with water to a predetermined volume. The sample is submerged in the water and the change in the volume of water is noted. The change in water volume reading (in cubic centimeters) is equal to the volume of the sample. The same number (in grams) is the weight of an equal volume of water. The average specific gravity is computed as follows:

$$G_{avg.} = \frac{\text{air-dry weight of sample}}{\text{weight of equal volume of water}}$$

(4) When a sample is too large to be submerged in a graduated cylinder, another method can be used. A can or container large enough to take the sample is placed in a pan that will catch any overflow. The can is filled with water to the top. The air-dried and weighed sample is submerged in the can and the overflow volume of water caught in the pan is measured. This value converted to grams is the weight of a volume of water equal to the volume of the sample. The formula for computing ($G_{avg.}$) is:

$$G_{avg.} = \frac{\text{weight of air-dried sample (in grams)}}{\text{weight of an equal volume of water (in grams)}}$$

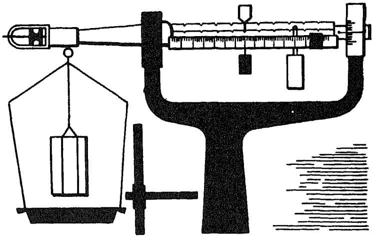


Figure 2–58. Expedient method of determining specific gravity: weighing in air.

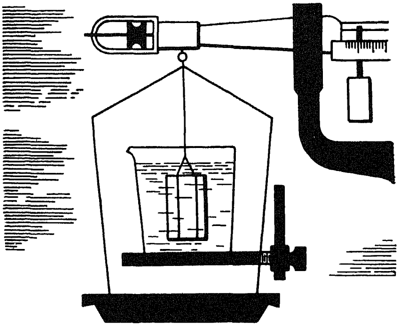


Figure 2–59. Expedient method of determining specific gravity: weighing in water.

Section V. HYDROMETER ANALYSIS

2–36. Description

a. Hydrometer analysis is based on Stokes' law which relates terminal velocity of a free-falling sphere in a liquid to its diameter. The relation is expressed by the following equation,

$$V = \frac{G_s - G_w}{18n} \quad D^2$$

where:

V = terminal velocity
G_s = specific gravity of solids
G_w = specific gravity of the liquid in which the sphere is falling
n = viscosity of the liquid
D = diameter of the sphere

It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various shapes and sizes. Larger particles settle more rapidly than the smaller ones. Beginning with a uniform dispersion of particles suspended through the liquid, after a time all particles larger than Diameter D will have settled below a depth H_R, where $H_R = Vt$. Application of Stokes' law permits calculation of grain sizes in silts and clays, assuming an equivalent spherical shape of particle.

b. The density of water with the soil particles dispersed in it depends upon the concentration of soil particles. If the suspension is allowed to stand, the soil particles will settle and the density will be decreased. The hydrometer is the instrument used to measure the density of the suspension at a known depth below the surface. The density measurement, together with knowledge of the specific gravity of the soil particles, determines the percentage of dispersed soil particles in suspension at the time and depth of measurement. The depth at which the measurement is made is found by calibrating the hydrometer. Stokes' law is used to calculate the maximum equivalent particle diameter for the material in suspension at this depth and for the elapsed time of settlement. A series of density measurements at known depth in the suspension and at known times of settlement thus provides determination of the percentages of particles finer than the diameter given by Stokes' law for each time. Thus, the series of readings will reflect the amount of different sizes of particles in the fine-grained soils.

2–37. Apparatus

a. Equipment for performing this test ((1) to (4) below) is not in the soil test set. The apparatus required includes:

(1) Hydrometer (calibrated at 20°C.—graduated in specific gravity; range 0.995 to 1.040, accuracy ±0.001; or graduated in grams per liter, range 0 to 50, accuracy ±1).

(2) Dispersion apparatus (or stirrer) to keep the soil particles dispersed.

(3) Sedimentation cylinder of glass (essentially 18 inches high, 2½ inches in diameter, and marked for a volume of 1000 ml.).

(4) Centigrade thermometer range 0° to 50° C., and accurate to 0.5 C.

(5) Timing device (watch or clock) with second hand.

(6) Balance, sensitive to 0.1 g.

(7) Oven.

b. A sheltered location for taking readings is required. The sedimentation cylinders must not be disturbed during the test. A steady bench or a level floor in a location having adequate light for taking readings should be provided.

2–38. Preparation

Prior to making the test, the hydrometer must be calibrated and a meniscus correction determined. A dispersion agent should be acquired.

a. Hydrometer Calibration. The hydrometer must be calibrated to determine its true depth in terms of the hydrometer reading.

(1) The volume of the bulb is determined either by measuring the volume of water displaced or by weighing the hydrometer. Since the specific gravity of the hydrometer is about 1, the weight in grams may be recorded as the volume in cubic centimeters.

(2) Compute the area of the graduate (A). Measure the distance between two graduations. The volume between two measured marks is divided by the measured distance to obtain the area.

(3) Measure the distance from the lowest graduation to the other major graduation (R).

(4) Measure the distance from the neck of the bulb to the lowest graduation. The distance H_1, corresponding to a reading, R, equals the sum of the two distances measured in (3) and (4).

(5) Measure the distance from the neck to the ip of the bulb. This value is h, the height of the bulb, d h/2 locates the center of volume of a symmetrical b. If a nonsymmetrical bulb is used, the center of volume can be determined with sufficient accuracy by projecting the shape of the bulb onto a sheet of paper and locating the center of gravity of the projected area.

(6) The measured values are substituted in the following formula:

$$H_R = H_1 + \frac{1}{2}\left(h - \frac{V_R}{A}\right)$$

where:

H_R = true height of suspension above the center of the hydrometer bulb (in cm).
H_1 = distance corresponding to the reading on the hydrometer (in cm).
h = height of hydrometer bulb (in cm).
V_R = volume of hydrometer (in cc).
A = inside area of graduate (in sq cm).

(7) By plotting the true height (H_R) against the reading R for the major graduations, a calibration curve can be drawn. This relation is essentially a straight line for a hydrometer having a streamlined shape.

b. Meniscus Correction. Hydrometers are calibrated to read correctly at the surface of the liquid. Soil suspensions are not transparent and the reading is made not at the surface but where the upper rim of the meniscus touches the glass. A correction for this meniscus (C_M) must be determined using clear distilled or demineralized water. This correction will be constant for a given hydrometer. The hydrometer is immersed in the water and the difference in height between the surface and the meniscus carefully determined. This value (C_M) is approximately 0.5, which may be used for routine work where greater accuracy is not required.

c. Preparation of Sample.

(1) The soil is separated on the No. 10 sieve, and the percentage finer than this sieve is determined. Samples for hydrometer analysis are taken from representative material finer than the No. 10 sieve.

(2) The approximate size of sample for hydrometer analysis varies according to the type of soil being tested:

Soil type	*Dry weight, g.*
Fat clays	30
Lean clays and silty soils	50
Sandy soils	75

(3) The exact dry weight of the sample in suspension must be determined either before or after the test. For economy of time and labor, it is preferable to oven-dry and weigh the sample before the test. In this case, any loss of material during the test will affect the results. Also, for some clays, drying may cause permanent changes in the grain sizes.

Samples of such soils should be preserved at natural water content, if possible, and tested before oven-drying. The dry weight is determined at the conclusion of the hydrometer analysis.

d. Dispersing Agent. Very fine soil grains in suspension normally tend to flocculate (adhere to each other in clusters) and to settle through the solution as a larger unit. This can lead to errors in the analysis.

(1) To prevent flocculation, a dispersing agent is added to the solution which keeps the individual particles separated. In most instances, 15 ml of a dispersing agent solution is adequate. An additional 15 ml can be added a second or third time if the flocculation continues. The following agents have been found satisfactory for most types of soil and are listed in an approximate order of effectiveness:

Dispersing agent	Stock solution	
	Concentration	g./liter
Sodium tripolyphosphate	0. 4N	29
Sodium polyphosphate	0. 4N	36
Sodium tetraphosphate (Quadrofos).*	0. 4N	31
Sodium hexametaphosphate (Calgon).**	0. 4N	41

*Trade names.

**Calgon is buffered with Na_2CO_3.

(2) Sodium silicate should not be used. Phosphate solutions are somewhat unstable and should not be stored for extended periods of time.

(3) A dispersing agent increases the liquid density and a correction must be made to the observed reading. The correction, (C_d), is determined by first adding the amount of dispersing agent to be used to a graduate partially filled with distilled or demineralized water. The graduate is then filled to the 1000 ml mark and a hydrometer reading taken. The correction factor, (C_d) is the difference between this reading and another taken in pure distilled or demineralized water.

e. Other Considerations.

(1) Temperature changes can effect a hydrometer test. The location for the test should be selected where temperature variations are at a minimum. The solution should be kept away from heat sources such as radiators and sunlight, and where temperatures can change such as near open windows.

(2) Highly organic soils require oxidation of organic matter before performing the hydrometer test. Oxidation is accomplished by mixing a solution of 30 percent hydrogen peroxide with the sample.

(3) If oven-dry weight is required before the test, the sample should be dried in the oven, allowed to cool to room temperature, and weighed to the nearest 0.1 g. This dry weight should be recorded.

2–39. Procedures

The hydrometer test consists of the following steps:

a. Record all identifying information for the sample (fig. 2–60). DD Form 1794 (Hydrometer Analysis) will be reproduced locally for Army use on 8″×10½″ paper (image size 7.4/10″×9 5/6″) (fig. 2–60). Supplies of DD Form 1794 for the other services will be requisitioned through normal publications supply channels.

b. Record the meniscus correction, (C_m), and the dispersing agent correction, (C_d), if an agent is to be used.

c. Determine or estimate the specific gravity of solids and record this value.

d. Place the sample in a numbered dish and add sufficient distilled or demineralized water until the sample is submerged. Add the dispersing agent. Allow the sample to soak overnight or until all soil lumps have disintegrated.

e. Add the oxidizing agent to the sample if it is highly organic. If only a small amount of organic matter is present, omit the oxidizing agent.

f. Transfer the soil-water slurry from the dish to a dispersing cup, washing (with distilled water) any residue in the dish into the cup. Add distilled water to the cup until the surface is 2 to 3 inches below the top. (Too much water will splash out during the mixing and carry out some of the soil sample.) Place the cup into the dispersing machine and mix for 1 to 10 minutes depending on the plasticity of the soil; the lower the plasticity, the shorter the mixing time.

g. Transfer the mixed suspension into a 1000-ml sedimentation cylinder and add distilled or demineralized water until the volume equals 1000 ml. At this time, the suspension should be brought to the temperature expected to prevail during the test.

h. One minute before starting the test, hold the graduate in one hand, and place the other hand or a suitable rubber cap over the open end to act as a stopper. Shake the suspension vigorously for a few seconds to raise any sediment from the bottom of graduate into suspension. Continue the agitation for the remainder of the minute by turning the cylinder upside down and back. In some cases, the sediment at the bottom may have to be loosened with a glass rod before shaking. Figure 2–61 illustrates a type of hand agitator which may be used instead of shaking. This agitator prevents the accumulation of sediment at the base and on the sides of the graduate. It is moved down through the suspension for the 1 minute.

HYDROMETER ANALYSIS

PROJECT LAKELAND BASE			DATE
BORING NO. STA 1 + 28	SAMPLE OR SPECIMEN NO. -	CLASSIFICATION CH	
DISH NO. 309	GRADUATE NO. 1724 A		HYDROMETER NO. 762
DISPERSING AGENT USED NONE			QUANTITY —
DISPERSING AGENT CORRECTION C_d =		MENISCUS CORRECTION C_m = 0.5	

TIME	ELAPSED TIME MIN	TEMP °C	HYDRO. READING (R')	CORRECTED READING (R)	PARTICLE DIAMETER (D), mm	TEMP CORRECTION (m)	$R + m$	PERCENT FINER	
								PARTIAL	TOTAL
9:35	0								
36	1	23.3	31.5	32.0	.041	+.6	32.6	100	
37	2	23.3	29.5	30.0	.030	.6	30.6	99	
39	4	23.3	29.0	29.5	.021	.6	30.1	97	
50	15	24.2	26.5	27.0	.011	.8	27.8	90	
10.05	30	26.7	23.0	23.5	.0080	1.4	24.9	81	
35	60	26.7	21.5	22.0	.0055	1.7	23.4	76	
11:35	120	27.6	19.0	19.5	.0040	1.6	21.1	68	
1:35	240	28.0	17.0	17.5	.0029	1.8	19.3	62	
9:35	1440	22.2	16.0	16.5	.0012	+0.4	16.4	55	

WEIGHT IN GRAMS		
Dish plus dry soil		59.4
Dish		10.0
Dry soil	W_o	49.4

Specific gravity of solids, G_s = 2.68

Corrected hydrometer reading (R) = hydrometer reading (R') + C_d + C_m

The particle diameter (D) is calculated from Stokes' equation using corrected hydrometer reading. Use nomographic chart for solution of Stokes' equation.

Hydrometer graduated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_o} (R + m)$$

Hydrometer graduated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_o} (R + m)$$

$$\text{Total percent finer} \doteq \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

W_s = total oven-dry wt of sample used for combined analysis

W_o = oven-dry wt in grams of soil used for hydrometer analysis

W_1 = oven-dry wt of sample retained on No. 200 sieve

REMARKS

TECHNICIAN	COMPUTED BY	CHECKED BY
J.P. Marks	J. P. Marks	C. A. Smith

DD FORM 1 FEB 71 **1794**

Figure 2–60. DD Form 1794 Hydrometer analysis data sheet, sample.

HYDROMETER ANALYSIS

PROJECT

DATE

BORING NO.

SAMPLE OR SPECIMEN NO.

CLASSIFICATION

DISH NO.

GRADUATE NO.

HYDROMETER NO.

DISPERSING AGENT USED

QUANTITY

DISPERSING AGENT CORRECTION C_d =

MENISCUS CORRECTION C_m =

TIME	ELAPSED TIME MIN	TEMP °C	HYDRO. READING (R')	CORRECTED READING (R)	PARTICLE DIAMETER (D), mm	TEMP CORREC-TION (m)	$R + m$	PERCENT FINER	
								PARTIAL	TOTAL

WEIGHT IN GRAMS	Dish plus dry soil		
	Dish		
	Dry soil	W_o	

Specific gravity of solids,

G_s =

Corrected hydrometer reading (R)

= hydrometer reading $(R') + C_d + C_m$

The particle diameter (D) is calculated from Stokes' equation using corrected hydrometer reading. Use nomographic chart for solution of Stokes' equation.

Hydrometer graduated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100}{W_o} (R + m)$$

Hydrometer graduated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_o} (R + m)$$

$$\text{Total percent finer} = \text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

W_s = total oven-dry wt of sample used for combined analysis

W_o = oven-dry wt in grams of soil used for hydrometer analysis

W_1 = oven-dry wt of sample retained on No. 200 sieve

REMARKS

TECHNICIAN

COMPUTED BY

CHECKED BY

DD FORM 1 FEB 71 1794

Figure 2-60. DD Form 1794 (Hydrometer Analysis)—Continued

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2–48

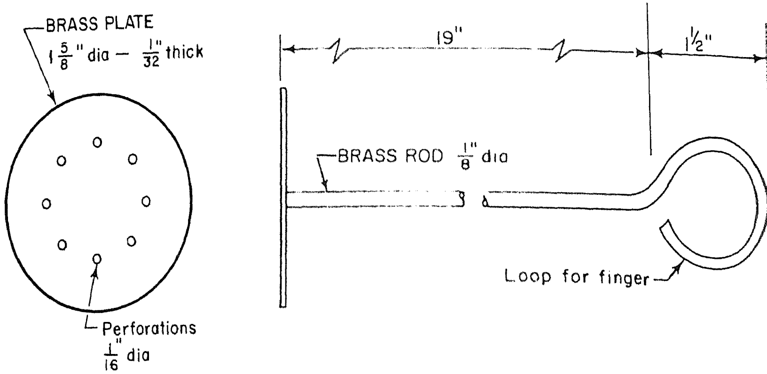


Figure 2–61. Hand agitator.

i. After 1 minute of agitation, set the cylinder on a table. If foam has developed on the surface of the suspension, remove it by touching it lightly with a piece of soap. Slowly immerse the hydrometer in the liquid, wait about 20 to 25 seconds, and take a reading. Care must be taken in inserting and removing the hydrometer to prevent disturbing the suspension.

j. Observe and record hydrometer readings after 1 and 2 minutes have elapsed from the time the cylinder was set down. Make all readings at the top of the meniscus. As each reading is completed, remove the hydrometer carefully and place it in a container of clean water. The soil material will settle on or adhere to the hydrometer if it is left in the suspension and cause an error in the readings.

k. Repeat the readings and record them after elapsed times of 4, 15, and 30 minutes and 1, 2, 4, and 24 hours. Remove the hydrometer from the suspension after each reading and place it in a graduate of clean water. When the hydrometer analysis is used for soil classification purposes, the 4- and 24-hour readings may not be required.

l. With hydrometers graduated to read grams per liter, record the actual reading. With hydrometers graduated to read specific gravity, read only the last two figures and estimate the third. Record the specific gravity indicated reading, minus 1.000, multiplied by 1000. *For example:* a reading of 1.0225 is recorded as 22.5 (1.0225−1.000=.0225; .0225×1000=22.5).

m. At the end of the 2-minute hydrometer reading and after each subsequent reading, place a thermometer in the suspension and record the temperature
to ±0.3° C. Temperature changes will affect th
results.

n. If the dry weight had not been determined,
fully wash all of the suspension into an evapor
dish. Oven dry the material, allow it to coo
determine the sample weight.

2–40. Computations

The recorded readings must be corrected an
verted into size distribution of the particles.

a. Corrected Hydrometer Readings (*R*). Alg
cally add the dispersing agent correction,
(para 2–38*d*) and the meniscus correction
(para 2–38*h*) to the actual hydrometer rea
(R′). The result is the corrected hydrometer
ings, (R).

b. Particle Diameter. The particle diameter
culated for each reading on the basis of Stokes
tion using the nomograph shown in figure 2–6
hydrometer calibration (para 2–38*a*) furnish
true height (H_R) on the hydrometer. The R-
designed for the particular hydrometer used
test. The nomograph contains the key which
the steps to follow in computing the diameter.
each reading. Place a straightedge (enginee
angle or transparent scale) on the specific gra
solids (point 1) and the temperature at the t
reading (point 2) and mark the intersection
line on the A-scale (point 3). Refer to the hy
eter calibration for the true height (H_R) and
on the scale (point 4). Using the straighted
nect point 4 and the time of reading (point 5
cate point 6, the velocity of settlement. Pl

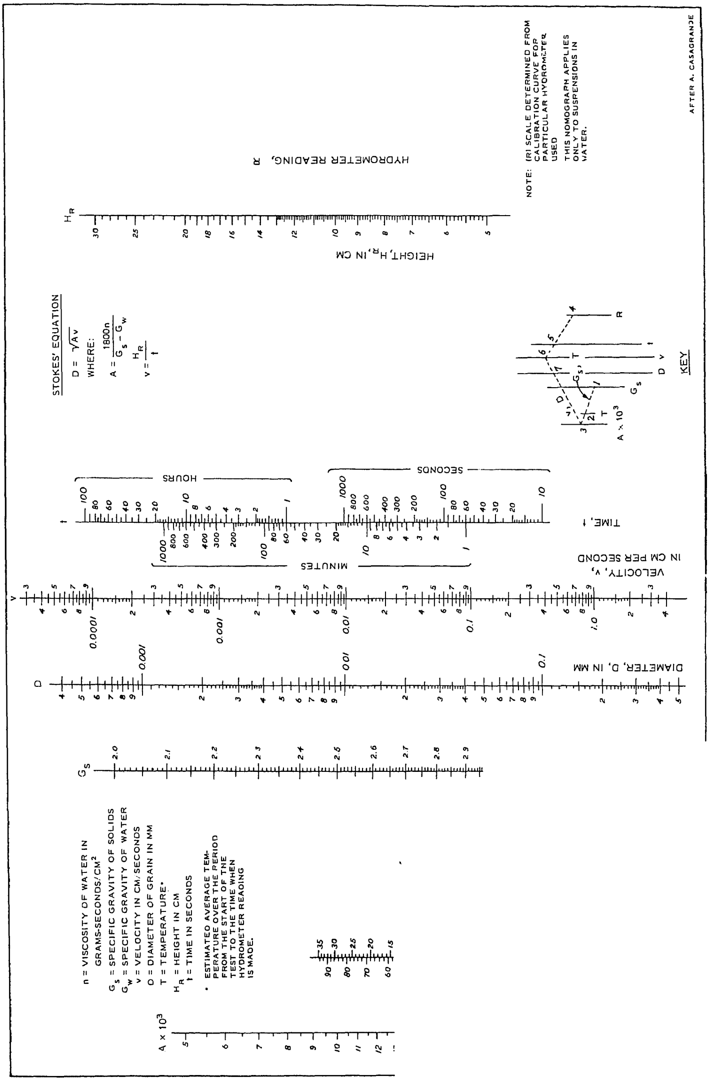

STOKES' EQUATION
D = √Av
WHERE:
A = 1800n / (Gs − Gw)
v = HR / t
n = VISCOSITY OF WATER IN GRAMS-SECONDS/CM²
Gs = SPECIFIC GRAVITY OF SOLIDS
Gw = SPECIFIC GRAVITY OF WATER
v = VELOCITY IN CM/SECONDS
D = DIAMETER OF GRAIN IN MM
T = TEMPERATURE*
HR = HEIGHT IN CM
t = TIME IN SECONDS
* ESTIMATED AVERAGE TEMPERATURE OVER THE PERIOD FROM THE START OF THE TEST TO THE TIME WHEN HYDROMETER READING IS MADE.
HYDROMETER READING, R
HEIGHT, HR, IN CM
HR
TIME, t
HOURS
MINUTES
SECONDS
VELOCITY, v, IN CM PER SECOND
DIAMETER, D, IN MM
Gs
A × 10³
T
KEY
NOTE: (R) SCALE DETERMINED FROM CALIBRATION CURVE FOR PARTICULAR HYDROMETER USED
THIS NOMOGRAPH APPLIES ONLY TO SUSPENSIONS IN WATER.
AFTER A. CASAGRANDE

Figure 2-62. Stokes' la

straightedge on points 3 and 6 and read the particle diameter (D) (point 7) in mm at the intersection of this line and the scale of diameters. Record diameter so found on the data sheet.

c. Percent Finer. The grain-size distribution is computed as a percent finer. To make the computation, the temperature correction factor (m), must be added. The temperature at each reading is used to determine the correction factor from table 2–2. This factor is added algebraically to the corrected hydrometer reading, and the total (R+m) is recorded. To complete the computation, the following formulas are used:

(1) *Hydrometer calibrated in specific gravity.*

Percent finer (by weight) =

$$\frac{G_s}{G_s-1} \quad \frac{100}{W_s} \quad \times (R+m)$$

(2) *Hydrometer calibrated in grams per liter.*

Percent finer (by weight) =

$$\frac{100}{W_s} \quad (R+m)$$

Where:

G_s = specific gravity of solids

W_s = oven-dry weight of soil (in grams)

R+m = corrected hydrometer reading plus temperature correction.

Table 2–2. Temperature Correction Factor, m, For Hydrometer Analysis

°C.	°F.	m	°C.	°F.	m
14.0	57.2	−0.9	24.0	75.2	+0.8
14.5	58.1	−0.8	24.5	76.1	+0.9
15.0	59.0	−0.8	25.0	77.0	+1.0
15.5	59.9	−0.7	25.5	77.9	+1.1
16.0	60.8	−0.6	26.0	78.8	+1.3
16.5	61.7	−0.6	26.5	79.7	+1.4
17.0	62.6	−0.5	27.0	80.6	+1.5
17.5	63.5	−0.4	27.5	81.5	+1.6
18.0	64.4	−0.4	28.0	82.4	+1.8
18.5	65.3	−0.3	28.5	83.3	+1.9
19.0	66.2	−0.2	29.0	84.2	+2.1
19.5	67.1	−0.1	29.5	85.1	+2.2
20.0	68.0	0.0	30.0	86.0	+2.3
20.5	68.9	+0.1	30.5	86.9	+2.5
21.0	69.8	+0.2	31.0	87.8	+2.6
21.5	70.7	+0.3	31.5	88.7	+2.8
22.0	71.6	+0.4	32.0	89.6	+2.9
22.5	72.5	+0.5	32.5	90.5	+3.0
23.0	73.4	+0.6	33.0	91.4	+3.2
23.5	74.3	+0.7	33.5	92.3	+3.3
			34.0	93.2	+3.5

2–41. Presentation of Results

Hydrometer analysis separates the fine particles of a soil into a grain-size distribution. The percentages can be plotted on a grain-size distribution chart (DD form 1207, fig. 2–54) using the particle diameters (D), and the percentages.

Section VI. DECANTATION OR WET MECHANICAL ANALYSIS

2–42. Principles

a. The grain-size distribution of a fine grained soil using hydrometer analysis obviously requires laboratory facilities and experienced personnel. The military engineer can identify the characteristics with greater accuracy using Atterberg limits or plasticity tests rather than grain-size distribution methods. However, when analyzing a soil for frost susceptibility, a wet mechanical analysis or decantation test may be used as an expediency to determine the percentage of material smaller than 0.02-mm in diameter. A soil is considered to be frost susceptible if it contains 3 percent or more by weight of particles smaller than 0.02-mm in diameter.

b. The principle of wet mechanical analysis is identical to the one used in hydrometer analysis. The basis is Stokes' law which expresses the rate of fall of a particle through a liquid. In general, sedimentation occurs when a suspended solid separates from a liquid and falls to the bottom. The viscosity of the liquid will affect the rate of fall and is a definite factor in the sedimentation process. Larger particles settle out faster than the smaller particles, and with similar sizes, the greater the difference in specific gravity between the solids and the liquid, the more rapid the settling.

c. If a single sphere is allowed to fall through a liquid of infinite extent, its velocity will increase under the influence of gravity. However, within a few seconds, a constant terminal velocity is reached and maintained as long as the conditions are unchanged. This is the relation expressed by Stokes' law (para 2–36*a*). For general application and simpler computations, certain assumptions are made. These introduce errors which may be self-compensating or may require empirical corrections. These errors are a result of the following factors:

(1) Particles are assumed to be truly spherical but they seldom if ever attain this shape.

(2) The extent of the liquid (Stokes' law is based on water) is not infinite and particles tend to influence neighboring particles. Also, the sides of the container holding the liquid present a restricting influence on particles in their proximity.

(3) An average specific gravity is used for the particles whereas some of them may vary considerably from this average.

(4) The temperature of the suspension must be assumed constant throughout its use which may not be true. An increase in temperature reduces the liquid's viscosity and increases the rate of fall.

2-43. Apparatus and Sample Preparation

a. The equipment needed for the test may be available in the soil test set or may have to be improvised.

b. The standard equipment, some of which is illustrated in figure 2-63, includes:

(1) Balance.
(2) Spatula.
(3) Electric or Coleman stove.
(4) Graduated cylinder, 100-cc capacity.
(5) Evaporating dish.
(6) Metal boxes, as required.
(7) Stopwatch.
(8) Pestle, rubber-tipped or ceramic.
(9) Thermometer.
(10) No. 10 and No. 200 sieves.
(11) Battery filler.

c. Other equipment which should be available includes—

(1) Stirrer, soil dispersion.
(2) Sedimentation cylinder.

d. When the equipment is not available and has to be improvised, the following are some of the methods that may be used:

(1) Rubber covered pestle—improvised by placing a portion of an inner tube over a hardwood handle.

(2) Sedimentation cylinder—improvised from a piece of 2-inch steel pipe about 2 feet long or from any other cylinder of about the same diameter (fig. 2-64). Note that the figure indicates welding or soldering the base. However, the end of the cylinder may be threaded and capped or, under expedient conditions, may be covered with overlapping layers of water resistant tape to form a watertight joint. The

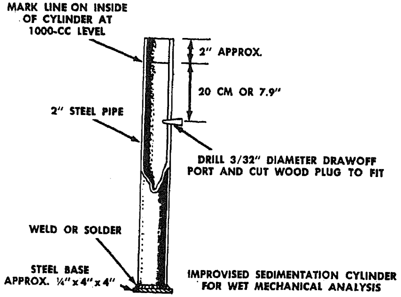

MARK LINE ON INSIDE OF CYLINDER AT 1000-CC LEVEL
2" APPROX.
20 CM OR 7.9"
2" STEEL PIPE
DRILL 3/32" DIAMETER DRAWOFF PORT AND CUT WOOD PLUG TO FIT
WELD OR SOLDER
STEEL BASE APPROX. ¼"x4"x4"
IMPROVISED SEDIMENTATION CYLINDER FOR WET MECHANICAL ANALYSIS

Figure 2–64. Improvised sedimentation cylinder.

plug for the $\frac{3}{32}$-inch hole in the wall may also be improvised by pointing a wood twig.

(3) Proceed as follows:

(*a*) Seal or cap one end.

(*b*) Pour 1000 cc of water into the cylinder and mark this level on the inside of the cylinder.

(*c*) Transfer and mark this same position on the outside. Measure 20 cm (7.9 inches) below this mark and drill a $\frac{3}{32}$-inch hole into the cylinder wall.

(*d*) Cut a wood or cork plug to fit the hole.

e. A deflocculating or dispersing agent (para 2–38*d*) is necessary to prevent the small particles from sticking to each other and yielding erroneous results. Prepare this solution in the proportion of 15 to 50 g of dispersing agent per liter of distilled water. The concentration of the solution should be higher for highly plastic soils.

f. The size of the sample should be proportional to the sand content of the soil. Select 30 to 50 g (dry weight) for clays and silts and 50 to 150 g (dry weight) for sandy soils. Since the test is made only on the portion of a sample which passes the No. 10 sieve, the sample must be separated using the No. 10 sieve. Determine the specific gravity of the sample. This will be used in the test.

2–44. Procedure, 0.02-mm Fraction

a. Oven-dry the sample that has passed through the No. 10 sieve, and cool and weigh it to the nearest 0.01 g. Record this weight (W_0).

b. Pulverize the dry sample with the rubber covered pestle. Do not crush the individual particles.

c. Transfer the sample to an evaporating dish and mix it with enough water to form a thin slurry.

d. Add 10 to 20 cc of dispersing solution, and allow the mixture to soak until all the particles are thoroughly separated. This should range from about 2 hours for a nonplastic soil to 24 hours for very plastic clays with high dry strength.

e. Transfer the soaked slurry to a dispersion stirrer cup, washing out the dish carefully with the battery filler to get all of the sample (fig. 2–65).

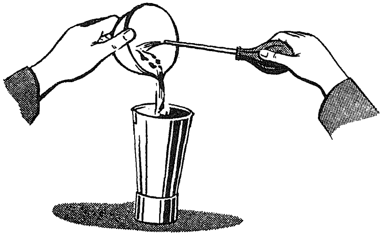


Figure 2–65. Transferring slurry to stirrer cup.

f. Stir the solution for not more than 1 minute. If a stirrer is not available, the mixing should be done by hand being careful not to spill any of the solution.

g. Transfer the mixed solution to the sedimentation cylinder, carefully washing all the material out of the cup with the battery filler (fig. 2–66). Add water to bring the mixture up to the 1000-cc mark and measure the temperature of the solution.

h. Using the palm of one hand as a stopper, shake the cylinder vigorously for 1 minute by reversing it end for end (fig. 2–67).

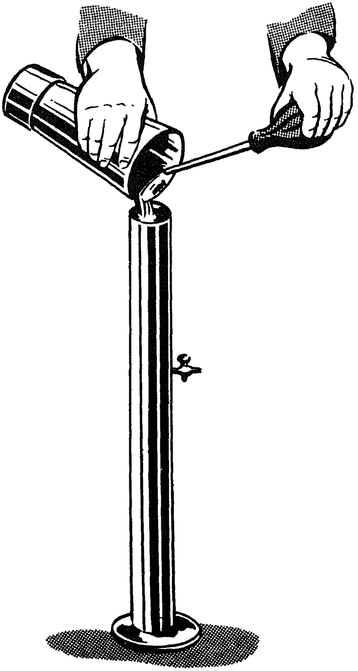


Figure 2–66. Transferring slurry to a sedimentation cylinder.

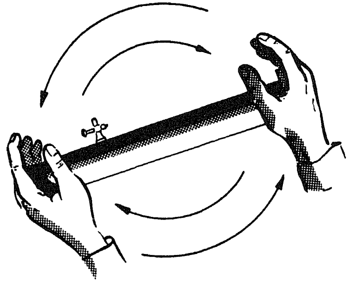


Figure 2-67. Shaking the mixture.

i. Set the cylinder on a firm base in a protected spot where it is not exposed to direct sunlight or other nonuniform source of heat.

j. Using the values for specific gravity and solution temperature as determined, select from table 2–3 the elapsed time for the 0.02-mm fraction to settle.

Table 2–3. Elapsed Time For Determining 0.02-mm Fraction.

Elapsed time (minutes)						
Water temperature		Specific gravity				
(° C.)	(° F.)	2.00 to 2.19	2.20 to 2.39	2.40 to 2.69	2.70 to 2.99	3.00 to 3.19
4. 5–9. 5	40–99	19	16	13	11	10
10. 0–15. 0	50–59	16	14	11	9	8
15. 5–20. 5	60–69	14	12	9	8	7
21. 0–26. 0	70–79	12	10	8	7	6
26. 5–31. 5	80–89	10	9	8	6	5
32. 0–37. 5	90–99	9	8	7	6	5

k. At the end of the elapsed time, measured by a stopwatch, remove the plug from the drawoff port and carefully collect 100 cc of suspension (fig. 2–68). Note that if a thorough mixing is assumed, grains larger than 0.02 mm in diameter will have settled through 20 cm of this solution. In the lower portions of the cylinder, these particles will have been replaced by similar particles settling from above. The top 20 cm, however, will not have the particles replaced and will only contain the smaller particles which have not settled during the elapsed time. Therefore, the collected 100 cc of solution has a sample of the original soil from which particles larger than 0.02 mm have been excluded.

l. Pour the 100 cc of suspension carefully into a metal box or evaporating dish (which has previously been weighed) and evaporate the water.

m. Weigh the residue and container to the nearest 0.01 g. Subtract the weight of the box or dish and record the result as the dry weight of the residue (W_1).

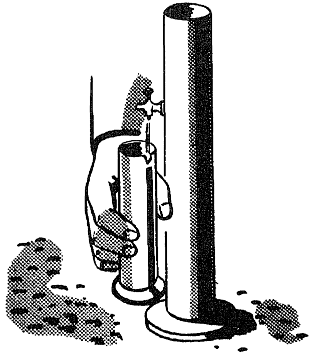


Figure 2–68. Drawing off sample from sedimentation cylinder.

2–45. Calculations and Corrections

a. To calculate the percent finer than 0.02-mm, a proportional relation of volume and weight is set up:

$$\frac{W_1}{V_1}=\frac{W_t}{V_t}$$

Where:

W_1 is the weight of the residue (para 2–44*m*).

V_1 is the measured drawoff (100 cc).

W_t is the weight of the fraction finer than 0.02-mm.

V_t is the measured total volume in the cylinder (1000 cc.).

The percentage of the total finer than 0.02-mm=

$$\frac{W_t}{W_o}\times 100$$

Where W_o is the weight of the sample passing through the No. 10 sieve (para 2–44*a*).

Example: Sieve analysis indicates that 25 percent of a soil sample passes the No. 10 sieve. 36.82g of the soil passing through the No. 10 sieve (W_o) is selected for the decantation test to determine if the soil is frost susceptible. After the test, the weight of the residue (W_1) is found to be 1.73 g. The percent finer is computed as follows:

$$\frac{W_1}{V_1}=\frac{W_t}{V_t}$$

$$\text{or } W_t=\frac{W_1}{V_1}\times V_t=\frac{1.73}{100}\times 1000=17.3\,g$$

$$\text{Percent finer}=\frac{W_t}{W_o}\times 100=\frac{17.3}{36.82}\times 100=47.0\%$$

Since only 25 percent of the entire sample passed the No. 10 sieve and of this 47 percent is finer than 0.02-mm, the percent finer referred to the entire soil sample is 47 percent of 25 percent =.47×.25=.1175, or

11.75%. This amount is greater than 3 percent (para 2–42*a*) and this soil can be considered frost susceptible.

b. Large variations in water temperature or specific gravity which can affect the results are corrected by reading the correct elapsed time before drawoff in table 2–3. Note that minor variations are blanketed within the range of values given for temperature or specific gravity. The average conditions are: 68° F., water temperature, and specific gravity of soil, 2.67. The elapsed time is given as 9 minutes. If either variable is off standard a small amount, it can be ignored. If large variations in temperature or specific gravity occur, the elapsed time before drawoff should be corrected to give a truer result.

2–46. Procedure for Other Grain Sizes

To complete a grain-size distribution curve of fine grained soil, several points on the curve must be determined and plotted. The wet mechanical analysis method can be used to establish these points.

a. Method. In general, three points are sufficient to plot the fine grain portion of the curve. These points are selected at the 0.05-mm, .02-mm, and .005-mm grain sizes. *A separate suspension for each desired point must be prepared.* The elapsed time for each size is determined from figure 2–69. This figure combines a nomograph of temperature and specific gravity to establish the first point on the left edge of the line graph. A line drawn through this point and parallel to the average condition line will establish the elapsed time to drawoff for the different particle diameters.

b. Example 1: To obtain elapsed time for a grain size smaller than 0.010-mm under average conditions—

(1) Follow the average condition line to its intersection with the 0.010-mm coordinate line.

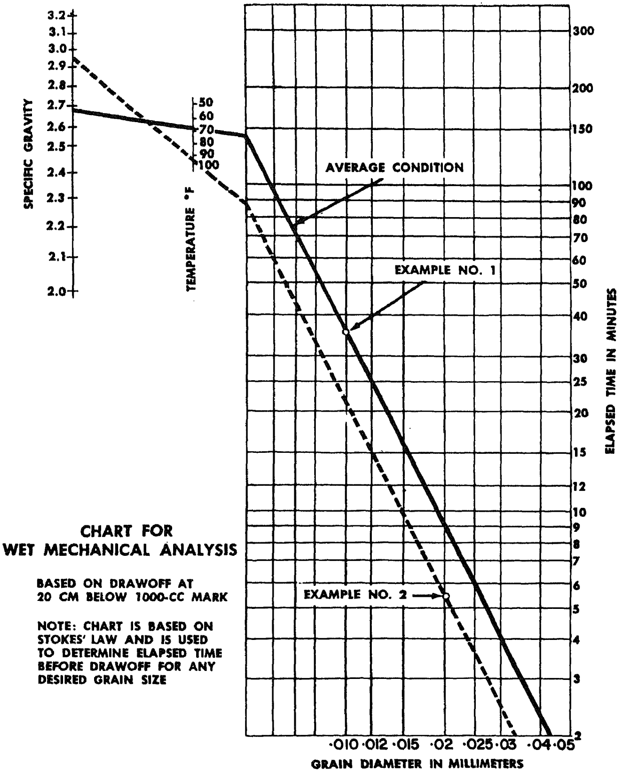


Figure 2–69. Wet mechanical analysis chart.

(2) Follow this value to the right-hand edge coordinates (between 30 and 40 minutes).

(3) The elapsed time to drawoff for the required size is about 35 minutes.

c. Example 2: To obtain elapsed time for grain sizes smaller than 0.02-mm, if specific gravity is 2.95 and the suspension temperature is 100°F.—

(1) Extend a line from 2.95 on the specific gravity scale through 100°F. on the temperature scale until it intersects the left-hand edge of the graph (at approximately 90 minutes).

(2) From this point, draw a line parallel to the average condition line (dashed line in figure 2-69).

(3) Follow this line until it intersects the 0.02-mm coordinate line.

(4) This value extended to the elapsed time coordinates on the right edge falls between 5 and 6 minutes.

(5) The elapsed time to drawoff is 5.5 minutes

Section VII. COMBINED MECHANICAL ANALYSIS

2-47. Description

Sieve analysis, hydrometer analysis, or wet mechanical analysis (secs. III, V, and VI of this chapter) are performed individually when the requirement for some individual characteristic, such as frost susceptibility or grain-size distribution, is specified. Many natural soils are mixed, that is, they contain both fine and coarse grained materials. Soil stabilization is accomplished in some cases by mixing finer graded soils with coarse sands and gravels. A complete grain-size analysis is needed to classify such mixed materials. To obtain a complete grain-size distribution curve of coarse and fine particles when there is a particular interest for both, *a combined mechanical analysis is conducted.* The analysis is performed in one of two ways depending on the method of determining the fraction finer than the No. 200 sieve, whether by hydrometer analysis or by sedimentation.

2-48. Apparatus and Sample Preparation (Hydrometer Analysis)

In this method, a sieve analysis and a hydrometer analysis are combined.

a. Apparatus. The apparatus for each of the two analyses is the same as used for the individual test. Sieve analysis equipment is described in paragraph 2-24 and hydrometer analysis apparatus in paragraph 2-37. The equipment should be assembled and checked prior to starting the analysis.

b. Sample Preparation. A representative sample is selected and prepared as for sieve analysis (para. 2-23). The size of the sample should be large enough to yield sufficient amounts of materials for both sieve and hydrometer analyses.

(1) A visual inspection of the sample will usually indicate the need for any intermediate steps such as large screen separation of sizes larger than the No. 4 sieve or prewashing to separate the sizes.

(2) Samples with fines of little or no plasticity are oven-dried, weighed, and separated on the No. 200 sieve. The plus and minus No. 200 sieve fractions are preserved for sieve and hydrometer analyses respectively.

(3) Soils containing plastic fines may be oven-dried initially. However, if the fines tend to adhere to the large particles and to form large lumps, the sample must be soaked in a pan with enough water to cover the material. Soaking must continue until all lumps and coatings have disintegrated. This will vary from 2 to 24 hours depending on the amount and plasticity of the fines. See paragraph 2-26.

(4) The water-soil mixture is washed over a No. 200 sieve (and a No. 4 sieve if necessary). The coarse fractions are preserved for sieve analysis, and the fines and water passing the No. 200 sieve are set aside for hydrometer analysis.

(5) Excess water with the fines is removed by evaporation, filtration, or wicking.

(6) If there is a possibility of altering grain-size of the fines by oven-drying, the dry weight measurement should be made after the hydrometer test.

2-49. Procedure (Hydrometer Test)

The combined analysis is performed as follows:

a. Record all identifying information for the sample on both the sieve analysis (fig. 2-52) and the hydrometer analysis (fig. 2-60) data sheets.

b. Perform a sieve analysis (para. 2-25) on the portion of the sample which was retained on the No. 200 sieve.

c. Perform a hydrometer analysis (para. 2-39) on the portion passing the No. 200 sieve.

d. Record all the test information on the appropriate data sheets.

e. The computations consist of the following steps:

(1) Compute the percentage of the total sample retained on the No. 200 sieve.

Percent retained on No. 200 sieve=

$$\frac{W_1}{W_s} \times 100$$

Where

W_1=dry weight of sample retained on the No. 200 sieve.

W_s=total dry weight of sample used for the combined analysis

(2) Compute the sieve analysis data as described in paragraph 2-27 except that the percent retained on each sieve is based only on that portion of the total material used for the sieve analysis, (W_2). The portion used may be less than the amount retained on the No. 200 sieve, and a partial percent retained must be computed:

Partial percent retained=

$$\frac{\text{Weight retained on a sieve}}{\text{Total weight used for sieve } (W_2)} \times 100$$

The total percent retained is computed as follows:

Total percent retained=

$$\text{Partial percent retained} \times \frac{W_1}{W_s}$$

The total percent finer=100—total percent retained.

(3) Compute the hydrometer analysis data as described in paragraph 2-40, except that the results are shown as a partial percent finer. As in the sieve analysis ((2) above), the amount of material used for hydrometer analysis may be less than the amount which passed the No. 200 sieve or less than (W_s)-(W_1). A partial percent finer is computed as follows:

Hydrometer calibrated in specific gravity

$$\text{Partial percent finer} = \frac{G_s}{G_s - 1} \times \frac{100 + (R+m)}{W_o}$$

Hydrometer calibrated in grams per liter

$$\text{Partial percent finer} = \frac{100}{W_o}(R+m)$$

Where:

W_0=oven-dry weight in grams of soil used for hydrometer analysis

G_s =specific gravity of solids

R =corrected hydrometer reading

m =temperature correction

The total percent finer is computed as follows:

Total percent finer=

$$\text{partial percent finer} \times \frac{W_s - W_1}{W_s}$$

2-50. Apparatus and Sample Preparation (Sedimentation Analysis)

In this type of analysis, a representative sample is separated on the No. 10 sieve, and the sedimentation analysis is performed on the portion passing through the sieve.

a. Apparatus. The apparatus for this test is the same as that used for the sieve analysis (para 2-24) and the wet mechanical analysis (para 2-43) with one exception. Additional pans may be required for sorting the sample into the wet and dry analysis fractions.

b. Sample Preparation. The size of the sample depends somewhat on the amount of material larger than ¼ inch. With only a small amount of material larger than ¼ inch, select about 200 g of dry material. For gravelly soils, select a representative sample that will provide both about 200 g of dry material finer than the ¼ inch sieve, and about 1000 g total material if the maximum particle size is in the range of ¾- to 1½-inch sieves.

2-51. Procedure (Sedimentation Test)

Recommended procedures for a combined sieve and wet mechanical analysis, using the sedimentation test (para 2-46), are given in the following paragraphs. Refer to the chart, figure 2-70, for the sequence of operations and to DD Form 1795 (Combined Mechanical Analysis (Sedimentation Test)), figure 2-71, for recording and computing results. DD Form 1795 will be reproduced locally for Army use on 8″x10½″ paper (image size 7⁴⁄₁₀″x9⅝″). Supplies of DD Form 1795 for the other services will be requisitioned through normal publications supply channels. Details of procedures for a combined sieve and wet mechanical analysis are as follows:

a. Oven-dry the selected sample at 110° C.±5° to a constant weight.

b. Cool the sample and weigh it to the nearest 0.1 g. Record this weight as the original weight of sample on the sieve analysis data sheet (fig. 2-71).

c. Pulverize or separate all fractions using a rubber pestle.

d. Soak the entire pulverized sample for 24 hours in about 500 cc of water mixed with a deflocculating solution (para 2-38*d*). Clean sands and gravels will only require about 2 hours of soaking.

e. Wash the soaked sample over a No. 10 sieve with a minimum of additional wash water. Make sure the sample is separated into individual particles and that none of the fine material is clinging to the particles retained on the sieve. Catch all the particles and wash water passing the No. 10 sieve.

f. Place the porton retained on the sieve in the oven and dry to a constant weight. Record its weight, then set it aside for later use.

g. Place the portion, including the wash water, which passed the No. 10 sieve in the oven and dry to a constant weight. Weigh and record the weight.

h. Select representative amounts of this dried portion for wet mechanical analyses (para 2-44 through 2-46). Save the remaining material for later use. Weigh each portion and record dry weights, (W_o), under sedimentation tests (fig. 2-71).

i. Soak the selected amount in water mixed with a dispersing agent and complete the mechanical analysis procedure as described in paragraph 2-44*d* to *m*. Record weights in the form for sedimentation analysis.

j. Compute partial percent finer than the selected particle diameters in each sedimentation test. Use formulas in paragraph 2-45. This is the percent finer in the material passing the No. 10 sieve. Compute the percent finer than the selected diameters for the total

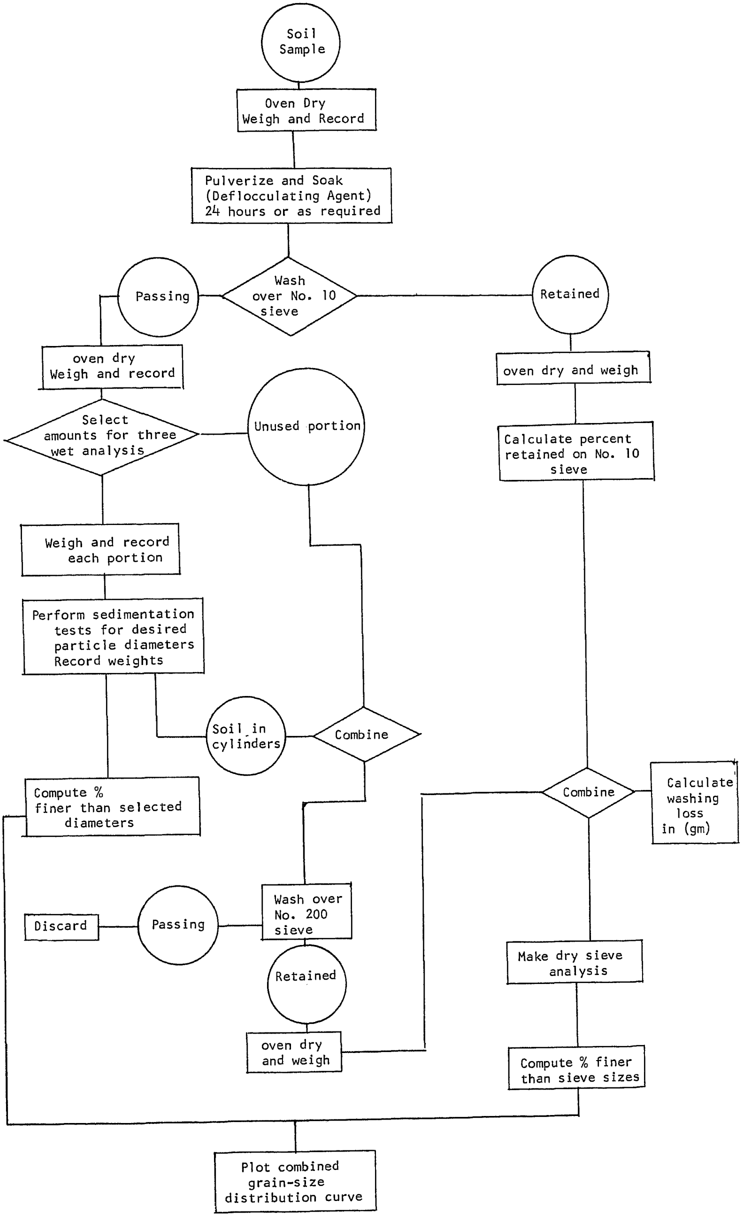

Soil Sample
Oven Dry Weigh and Record
Pulverize and Soak (Deflocculating Agent) 24 hours or as required
Wash over No. 10 sieve
Passing
Retained
oven dry Weigh and record
oven dry and weigh
Select amounts for three wet analysis
Unused portion
Calculate percent retained on No. 10 sieve
Weigh and record each portion
Perform sedimentation tests for desired particle diameters Record weights
Soil in cylinders
Combine
Compute % finer than selected diameters
Combine
Calculate washing loss in (gm)
Discard
Passing
Wash over No. 200 sieve
Retained
oven dry and weigh
Make dry sieve analysis
Compute % finer than sieve sizes
Plot combined grain-size distribution curve

Figure 2-70. Combined mechanical analysis (sedimentation method), operations sequence.

COMBINED MECHANICAL ANALYSIS *(Sedimentation Test)*		DATE
PROJECT	EXCAVATION	SAMPLE NUMBER

DESCRIPTION OF SAMPLE

WEIGHT TOTAL SAMPLE	WEIGHT PASSING NO. 10 SIEVE	PERCENT PASSING NO. 10 SIEVE

WEIGHT RETAINED ON NO. 10 SIEVE *(g)*	WEIGHT RETAINED ON NO. 200 SIEVE *(g)*	TOTAL WEIGHT FINER THAN NO. 200 SIEVE *(g)*	WASHING LOSS *(g)*

SIEVE ANALYSIS

SIEVE OR SCREEN	SIZE OF OPENING *(mm)*	WEIGHT OF SIEVE *(g)*	WEIGHT OF SIEVE AND SAMPLE *(g)*	WEIGHT RETAINED ON SIEVE *(g)*	CUMULATIVE WEIGHT RETAINED *(g)*	PERCENT RETAINED	PERCENT FINER
2 in.	50.8						
1-1/2 in.	38.1						
1 in.	25.4						
3/4 in.	19.0						
1/4 in. *(No. 3)*	6.35						
No. 4	4.76						
No. 10	2.00						
No. 40	0.420						
No. 60	0.250						
No. 100	0.149						
No. 200	0.074						
Pan							
Wash loss							

SEDIMENTATION ANALYSIS

SELECTED DIAMETER *(mm)*	DRY WEIGHT W_o *(g)*	WEIGHT OF RESIDUE PLUS DISH *(g)*	WEIGHT OF DISH *(g)*	DRY WEIGHT OF RESIDUE W_1 *(g)*	TOTAL WEIGHT FINER W_t *(g)*	PARTIAL % FINER THAN SELECTED DIAMETER	PERCENT FINER TOTAL SAMPLE
0.05							
0.02							
0.005							

TECHNICIAN	COMPUTED BY	CHECKED BY

DD FORM 1 FEB 71 **1795**

Figure 2–71. DD Form 1795 (Combined mechanical analysis (sedimentation method)), data sheet.

sample by multiplying the partial percents finer by the percent passing the No. 10 sieve, (P_1) above.

Example: If the percent finer than 0.02-mm of the passing No. 10 fraction (*g* above) was 27 percent and the combined dry sieve analysis resulted in 15 percent passing the No. 10 sieve, the percent finer than 0.02-mm for the entire sample would be:

$$\text{Percent finer} = \frac{27\% \times 15\%}{100} = 4.05 \text{ percent}$$

k. Combine the fraction passing the No. 10 sieve that was set aside (*h* above) with the soil left in the sedimentation cylinder, and wash over a No. 200 sieve to remove the fines.

l. Oven-dry at 100° C.±5° to constant weight the material retained on the No. 200 sieve after washing.

m. Combine the material retained on the No. 200 sieve (*l* above) with the portion retained on the No. 10 sieve (*f* above). Add the weights retained on No. 10 and on No. 200 sieves; subtract this sum from the original total weight of sample to get the washing loss.

n. Perform dry sieve analysis on the combined material retained on the No. 200 sieve. See instructions in paragraph 2-25 for procedure and paragraph 2-27 for calculations.

2-52. Presentation of Results

a. The combined mechanical analysis results in the percent finer by weight in terms of the different size particles in the soil sample. The tabulated results are useful for comparison to a set of specifications. An easily interpretable form is the grain-size distribution chart (fig. 2-54). The values of percent finer per grain size are plotted on a semilogarithmic chart. The plotted points are joined by a smooth curve. This curve can be used to determine the uniformity coefficient (C_u) and the coefficient of curvature (C_c). See paragraph 2-27*c* for the definitions of these terms.

b. The data in the example shown on the data sheet of figure 2-71 are plotted on DD Form 1207 (fig. 2-72), to give an example of such a curve for a mixed soil. For this soil, the diameter corresponding to 60 percent passing (D_{60}) is 1.2-mm and that for 10 percent passing (D_{10}) is 0.11-mm. Hence the uniformity coefficient is

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.2}{0.11} = 10.9$$

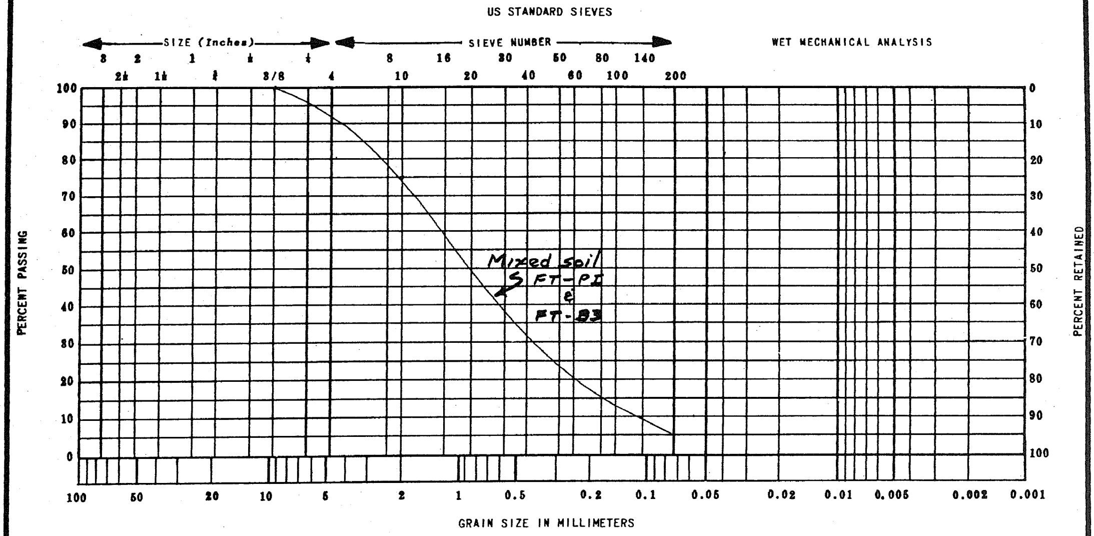
The diameter for 30 percent passing (D_{30}) is 0.4-mm. Thus, the coefficient of curvature is

$$C_c = \frac{(D_{30})^2}{(D_{60})(D_{10})} = \frac{(0.4)^2}{(1.2)(0.11)} = 1.21$$

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 17 JUN 69

PROJECT FRAUNCE THEATER



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS

TECHNICIAN (Signature) Paul Masston

PLOTTED BY (Signature) Paul Masston

CHECKED BY (Signature) L.G. Jones

DD FORM 1 AUG 57 1207

GPO 943037

Section VIII. IN-PLACE MOISTURE CONTENT TESTING

2–53. Description

a. Definition. Moisture content (w) is the ratio, expressed as a percentage, of the weight of water in in a soil mass to the weight of the soil matter (dry weight);

$$w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100.$$

b. Need for In-place Determinations. With many soils, close control of moisture content during field compaction by rolling is necessary in order to develop a required density and strength in the soil mass. The amount of compaction effort that must be exerted to obtain a specified density is dependent upon having the moisture content at, or very close to, optimum. Because the specified density is in terms of dry unit weight, the moisture content must be determined in conjunction with the wet unit weight in order to determine whether moisture must be added or whether moisture must be removed from the in-place soil to achieve the optimum moisture content. This is a necessary field procedure in the construction of embankments and compaction of highway subgrades. Adjustment of moisture during construction can be done only if the moisture content is known promptly. The time required for oven-drying often cannot be afforded.

c. Methods. Experienced engineers who become familiar with soils in an area can judge moisture content by field examination. The best and most accurate method uses an oven with temperature control. If time is not a consideration, the soil may be air-dried in the sun. Under expedient conditions, the soil sample may be dried in a frying pan or container heated by an external source. Because organic material may be burned off by high temperatures, an error may result. Another expedient method is to saturate the soil sample in alcohol and to burn the alcohol off. However, this procedure must be done carefully and repeated several times to achieve the best results. Also, the speedy moisture tester, a component of the soil test set, can be used.

2–54. Procedure and Computations

a. Apparatus. The apparatus needed for the laboratory method is as follows:

(1) Oven, preferably automatically controlled for 105°–110° C.

(2) Balance, sensitive to 0.01 g.

(3) Desiccator (unnecessary if samples are weighed immediately after cooling).

(4) Sample containers, with lids.

b. Test Procedure. The determination is accomplished by the oven or alcohol method in the following manner:

(1) Record identifying information of the sample on the data sheet, DD Form 1205 (fig. 2–73).

(2) Weigh and record the weights of the containers to be used in the test.

(3) Place the sample in a container, weigh it, and record the total weight of container (tare) and wet soil. If the weight of the sample is less than 100 g, weigh it to the nearest 0.1 g, otherwise the weight should be to the nearest 1.0 g.

(4) Place the open container and sample in an oven heated to 110°±5° C. and dry until the sample weight becomes constant. This will generally require 8 to 12 hours.

(5) Remove the container from the oven and place it in the desiccator to cool. If the container is too large for the desiccator, allow the sample to cool to room temperature in the open air.

(6) After the sample has cooled, weigh it and the container and record the weight (tare and dry soil) on the data sheet.

c. Computations.

(1) The moisture content (in percent) is equal to

$$w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

$$= \frac{\text{weight of wet soil} - \text{weight of dry soil}}{\text{weight of dry soil}} \times 100$$

$$= \frac{W_w}{W_s} \times 100$$

(2) The computations are made on the data sheet (fig. 2–73). Item A minus item B equals the weight of water (W_w). Item B minus item D equals the weight of dry soils (W_s). The division is made, converted to a percentage, and recorded as the water content (w).

(3) *Example:* A pan weighing 44.0 g was used to contain a sample of soil for determination of its moisture content. The total weight, including the pan, was 189.39 and after drying was reduced to 170.0 grams. What is the resulting percent moisture in the soil?

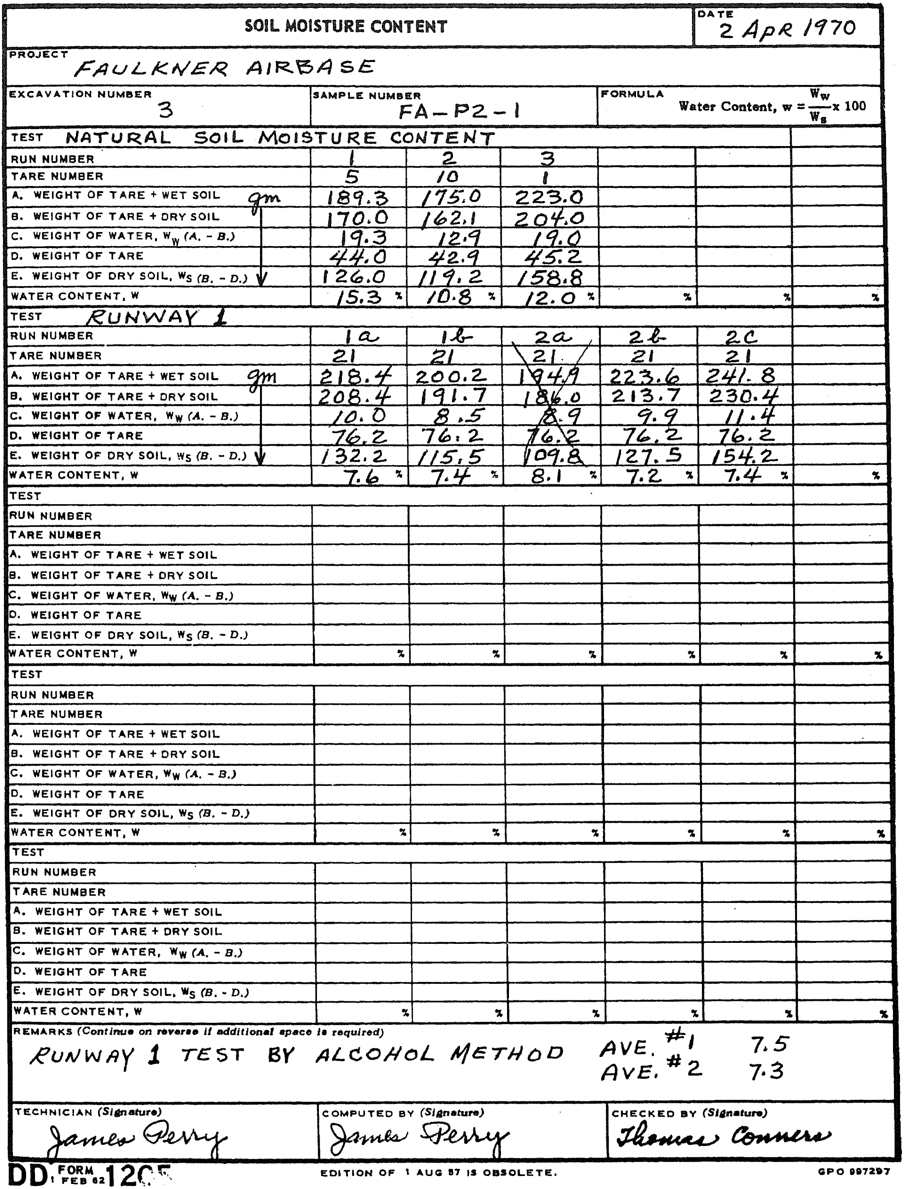
(*a*) Weight of water = (weight of wet sample plus tare) — (weight of dry sample plus tare) = 189.3 − 170.0 = 19.3 grams.

(*b*) Weight of dry soil = (weight of dry sample plus tare) — (weight of tare) = 170.0 − 44.0 = 126.0 grams.

(*c*) Water content =

$$\frac{100 \times \text{weight of water}}{\text{weight of dry soil}} = 100 \times \frac{W_w}{W_s}$$

$$w = 100 \times \frac{19.3}{126.0} = 15.3 \text{ percent.}$$



SOIL MOISTURE CONTENT	DATE 2 APR 1970
PROJECT FAULKNER AIRBASE	

EXCAVATION NUMBER 3	SAMPLE NUMBER FA–P2–1	FORMULA Water Content, $w = \frac{W_w}{W_s} \times 100$

TEST NATURAL SOIL MOISTURE CONTENT						
RUN NUMBER	1	2	3			
TARE NUMBER	5	10	1			
A. WEIGHT OF TARE + WET SOIL gm	189.3	175.0	223.0			
B. WEIGHT OF TARE + DRY SOIL	170.0	162.1	204.0			
C. WEIGHT OF WATER, W_W (A. – B.)	19.3	12.9	19.0			
D. WEIGHT OF TARE	44.0	42.9	45.2			
E. WEIGHT OF DRY SOIL, W_S (B. – D.)	126.0	119.2	158.8			
WATER CONTENT, W	15.3 %	10.8 %	12.0 %	%	%	%
TEST RUNWAY 1						
RUN NUMBER	1a	1b	~~2a~~	2b	2c	
TARE NUMBER	21	21	~~21~~	21	21	
A. WEIGHT OF TARE + WET SOIL gm	218.4	200.2	~~194.9~~	223.6	241.8	
B. WEIGHT OF TARE + DRY SOIL	208.4	191.7	~~186.0~~	213.7	230.4	
C. WEIGHT OF WATER, W_W (A. – B.)	10.0	8.5	~~8.9~~	9.9	11.4	
D. WEIGHT OF TARE	76.2	76.2	~~76.2~~	76.2	76.2	
E. WEIGHT OF DRY SOIL, W_S (B. – D.)	132.2	115.5	~~109.8~~	127.5	154.2	
WATER CONTENT, W	7.6 %	7.4 %	~~8.1~~ %	7.2 %	7.4 %	%
TEST						
RUN NUMBER						
TARE NUMBER						
A. WEIGHT OF TARE + WET SOIL						
B. WEIGHT OF TARE + DRY SOIL						
C. WEIGHT OF WATER, W_W (A. – B.)						
D. WEIGHT OF TARE						
E. WEIGHT OF DRY SOIL, W_S (B. – D.)						
WATER CONTENT, W	%	%	%	%	%	%
TEST						
RUN NUMBER						
TARE NUMBER						
A. WEIGHT OF TARE + WET SOIL						
B. WEIGHT OF TARE + DRY SOIL						
C. WEIGHT OF WATER, W_W (A. – B.)						
D. WEIGHT OF TARE						
E. WEIGHT OF DRY SOIL, W_S (B. – D.)						
WATER CONTENT, W	%	%	%	%	%	%
TEST						
RUN NUMBER						
TARE NUMBER						
A. WEIGHT OF TARE + WET SOIL						
B. WEIGHT OF TARE + DRY SOIL						
C. WEIGHT OF WATER, W_W (A. – B.)						
D. WEIGHT OF TARE						
E. WEIGHT OF DRY SOIL, W_S (B. – D.)						
WATER CONTENT, W	%	%	%	%	%	%

REMARKS (*Continue on reverse if additional space is required*)

RUNWAY 1 TEST BY ALCOHOL METHOD AVE. #1 7.5
AVE. #2 7.3

TECHNICIAN (*Signature*) James Perry	COMPUTED BY (*Signature*) James Perry	CHECKED BY (*Signature*) Thomas Conners

DD FORM 1 FEB 62 **1205** EDITION OF 1 AUG 57 IS OBSOLETE. GPO 997297

Figure 2–73. Soil moisture content data sheet.

2-55. Speedy Moisture Tester

a. The procedure for determining moisture content using the *speedy moisture tester* (*26 grams*) is as follows:

(1) Weigh a gram sample of soil.

(2) Place the soil sample and two 1¼-inch steel balls in the large chamber.

(3) Place three scoops of reagent in the cap. Then with the pressure vessel in a horizontal position, insert the cap into the pressure vessel, and tighten the clamp to seal the cap to the unit.

(4) Raise the moisture tester to a vertical position so that the reagent falls into the vessel.

(5) Holding the moisture tester horizontally, manually rotate the device for 10 seconds so that the steel balls are put into orbit around the inside circumference, and then rest for 20 seconds. Repeat the shake-rest cycle for a total of 3 minutes. Do not allow the steel balls to fall against either the cap or orifice leading to the dial, since this might cause damage.

(6) Read the pressure gage of the moisture tester and determine the moisture content of the soil on a dry-weight basis from the conversion chart. Record the moisture content.

(7) When sample is dumped, examine it for lumps. If the soil sample is not completely broken down, increase the time limit (shaking unit) by 1 minute on the next test.

b. Speedy Moisture Tester (*6 grams*). See instructions contained in the apparatus.

Section IX. LIQUID AND PLASTIC LIMITS

2-56. Discussion

a. Clays and some other fine-grained soils exhibit the physical property of plasticity if the proper amount of water is present in the soil (para 2-2*g*). A plastic soil is one that can be deformed beyond the point of recovery without cracking or change in volume. Such soils can be remolded. The liquid limit (LL) is the greatest water content which the material may contain and still remain plastic; more water causes it to become a thick liquid. The plastic limit (PL) is the lowest water content which the material may contain for plastic behavior; with less water the soil is semisolid and breaks into fragments if remolding is attempted. The plasticity index (PI) is the numerical difference between the liquid and plastic limits: (PI) = (LL) − (PL). A large PI indicates a very plastic clay, a small one denotes a clay having little plasticity. As water content decreases below the plastic limit, the semisolid clay mass shrinks and becomes stiffer. The shrinkage limit is the water content where further drying shrinkage ceases and below this water content the soil behaves as a solid. Since there is no sharp distinction between the semiliquid, plastic, and semisolid states of consistency, arbitrary, standardized procedures have been established for determining the liquid and the plastic limits.

b. Research with large numbers of clay soils was used to establish the soil *plasticity chart* for laboratory classification of fine-grained soils (table B-1, app B) that is part of the *Unified Soil Classification system*. Values of liquid limit and plasticity index are coordinates that locate a particular soil sample on the chart. The region on the chart in which the sample falls gives the classification, based upon the behavioral characteristics of the particular soil.

2-57. Liquid Limit Test Preparation

a. Definition. The liquid limit of a soil is the water content at the boundary between the liquid and the plastic states expressed as a moisture content and reported as a whole number. This boundary is arbitrarily defined as the water content at which two halves of a soil cake placed in a standard cup and divided into two sections by a standard grooving tool, will flow together for a distance of 0.5 inch along the bottom of the groove when the cup is dropped 25 times for a distance of 1 cm (0.3937 in.) at the rate of two drops per second.

b. Apparatus. The apparatus for a liquid limit test consists of the following:

Balance
Boxes, stainless steel, 2-inch diameter by ⅞ inch high
Dishes, evaporating
Filler, battery
Liquid limit machine, complete with brass cup and grooving tool
Mortar, porcelain
Oven, electric
Pan, sieve, 8-inch frame
Pestle, rubber-tipped
Screwdriver
Sieve, testing, 8-inch frame full height, U.S. No. 40
Spatula
Tongs, crucible

Some of this equipment is shown in figure 2-74.

c. Preparation. Soil samples should be at natural water content when preparing for the test. Although the effects of drying may be negligible for many soils, it is significant for some and any drying which occurs before testing will reduce the limit values. Certain

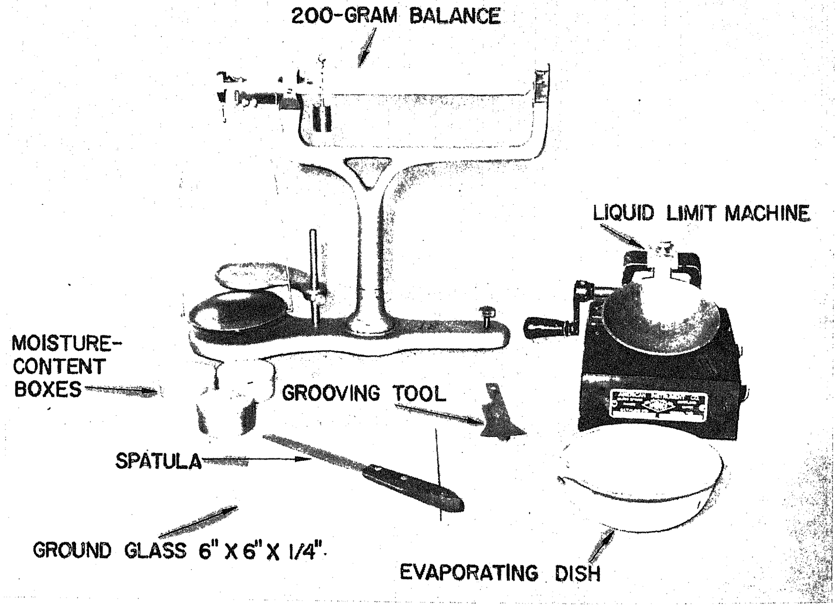


Figure 2–74. Apparatus for determining Atterberg limits.

soil colloids undergo an irreversible change on oven or complete drying. Liquid and plastic limit tests on these soils after drying will yield improper results. Factors which must be considered before testing are—

(1) The liquid limit is performed on material finer than the No. 40 sieve.

(2) Samples should be large enough to produce 150 to 200 grams of material for testing.

(3) The selected sample must not be subjected either to air-drying or oven-drying before testing.

(4) If the sample contains no material coarser that the No. 40 sieve, it should be thoroughly mixed. It will then be ready for testing.

(5) If the sample contains material coarser than the No. 40 sieve, it is soaked in clean water for 24 hours and washed over a No. 40 sieve using a minimum of wash water. The fines and wash water are caught in a large dish or collecting pan and saved.

(6) Material retained on the No. 40 sieve is oven-lried at 110°±5° C., and then dry-sieved through he No. 40 sieve.

(7) The portion that passed through the No. 40 ieve during the dry-sieving is combined with the ash-through portion for testing.

(8) The combined (passing No. 40) sample is dried to a putty-like consistency by decantation or evaporation being careful to prevent caking or lumping during the process.

(9) Neither chemical substances nor dry soil are added to hasten settlement nor to speed drying.

(10) The soil sample should be thoroughly mixed just before starting the test.

d. Checking and Adjusting the Liquid Limit Device. The liquid limit testing machine (fig. 2–75) must be inspected prior to the test to determine that it is in good working condition.

(1) Check the pin connecting the cup for wear. It must not be worn to the point where it permits sideplay of the cup.

(2) Check the screws connecting the cup to the hanger arm. They should be tight.

(3) Check the cup for wear. Long usage will develop a groove through the cup. This can be felt by running the fingers over the inside surface. If worn to this extent, it should be replaced.

(4) Check the grooving tool for wear. The dimensions which control the size of the groove are shown in figure 2–75. The tool should be discarded when the point width exceeds 0.086 inch.

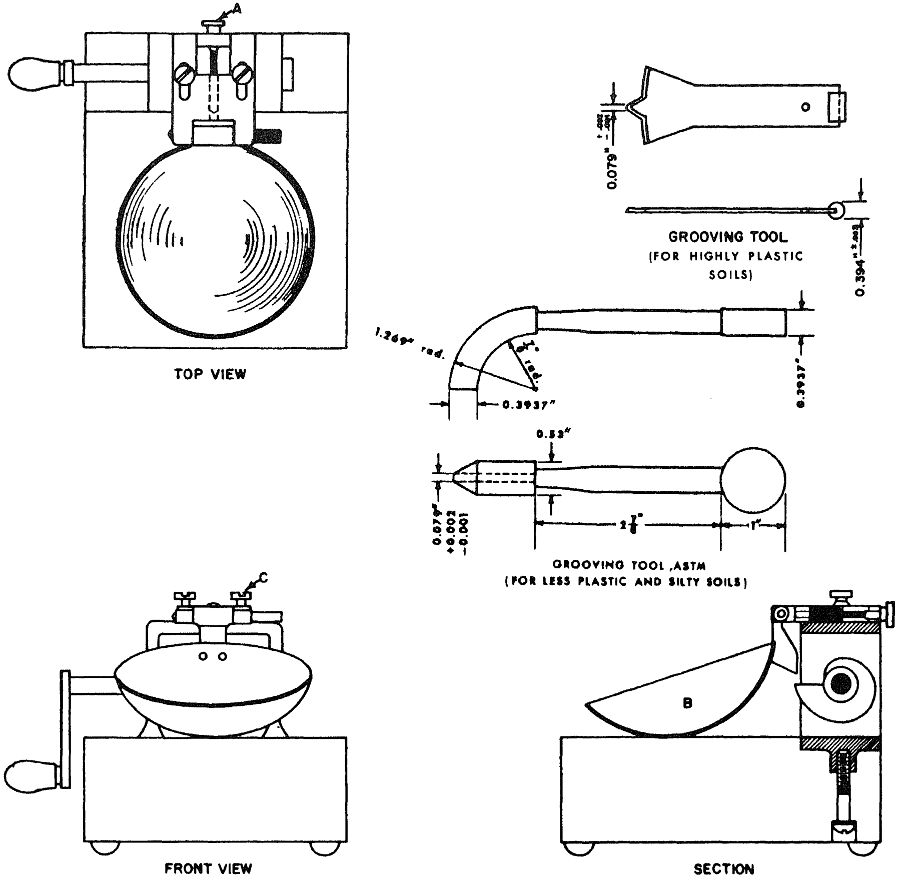


Figure 2–75. Mechanical liquid limit device.

(5) Each time the test is run, or at proper intervals, verify the height of the drop of the cup. The gage for this check is on the handle of the grooving tool. Using the gage, the height is adjusted by means of the adjusting screw (A, fig. 2–75) until the point on the cup that strikes the base (B) is exactly 1 cm (0.394 in.) above the base. Tighten screws (C) to secure the adjustment and check it by turning the crank at approximately two revolutions per second. If the adjustment is correct, a slight click will be heard. If the cup is lifted off the gage or no sound is heard, further adjustment is necessary.

2–58. Liquid Limit Test Procedure

With the sample prepared and the device checked, the test procedure consists of the following steps:

a. Record all identifying information for the sample on the top section of the data sheet, DD Form 1209 (fig. 2–76).

b. Place 50 to 80 grams of the prepared sample in the cup and level it off to a depth of approximately 1 cm at the point of greatest thickness (fig. 2–77). When leveling the soil, squeeze it downward and spread it using as few strokes of the spatula as possible. Be careful to prevent entrapping air bubbles in the mass. Return unused soil to sample stock.

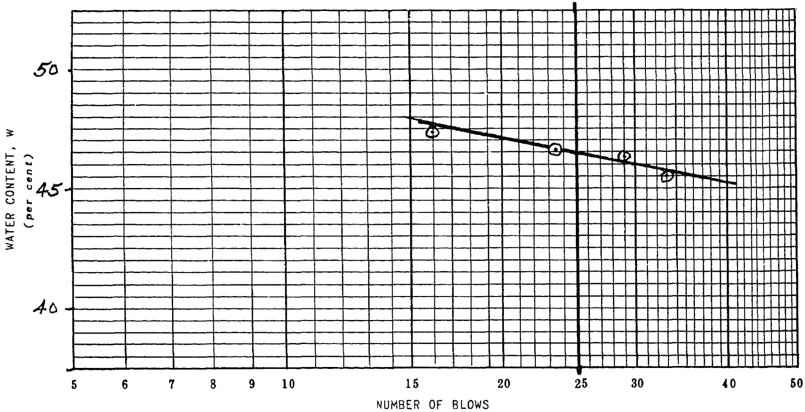
ATTERBERG LIMITS DETERMINATION	DATE 2 APR 1970

PROJECT BRITT AIRFIELD	EXCAVATION NUMBER 2+81	SAMPLE NUMBER B-P3-1

LIQUID LIMIT, w_L

RUN NUMBER	1	2	3	4		
TARE NUMBER	41	42	43	44		
A. WEIGHT OF WET SOIL + TARE	44.41	44.89	41.07	41.27		
B. WEIGHT OF DRY SOIL + TARE	41.08	41.52	38.16	38.39		
C. WEIGHT OF WATER, W_w (A.-B.)	3.33	3.37	2.91	2.88		
D. WEIGHT OF TARE	34.04	34.29	31.88	32.06		
E. WEIGHT OF DRY SOIL, W_s (B.-D.)	7.04	7.23	6.28	6.33		
WATER CONTENT, $w=(\frac{W_w}{W_s} \times 100)$	47.3	46.6	46.3	45.5		
NUMBER OF BLOWS	15-17-16	23-24	29-28-30	33-33		

w_L LL = 46	w_P PL = 18	I_P (w_L-w_P) PI = 28



PLASTIC LIMIT, w_P

					NATURAL WATER CONTENT
RUN NUMBER	1	2	3		
TARE NUMBER	45	46	47		48
F. WEIGHT OF WET SOIL + TARE	56.21	55.15	60.60		58.72
G. WEIGHT OF DRY SOIL + TARE	55.90	54.89	60.10		57.82
H. WEIGHT OF WATER, W_w (F.-G.)	0.31	0.26	0.50		0.90
I. WEIGHT OF TARE	54.10	53.40	57.43		55.02
J. WEIGHT OF DRY SOIL, W_s (G.-I.)	1.80	1.49	2.67		2.80
WATER CONTENT, $w=(\frac{W_w}{W_s} \times 100)$	17.2	17.4	18.7		32.1
PLASTIC LIMIT, I_P (Average w)			18		

REMARKS

Casagrande grooving tool used in liquid limits (LL) test.

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)

DD FORM 1 AUG 57 1209

GPO-831499

Figure 2–76. Data sheet, Atterberg limits determination.

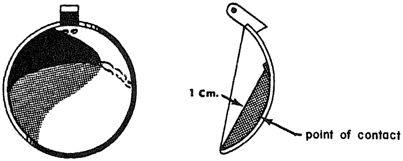


Figure 2-77. Leveling sample in the cup.

c. Holding the grooving tool and the cup as shown in figure 2-78, make the groove by drawing the tool down through the sample along the diameter through the centerline of the cam follower so that a clean, sharp groove of the proper dimensions will be formed.

(1) More than one stroke may be necessary to make the groove. Each stroke should be a little deeper than the preceding one until the last stroke scrapes the bottom of the cup clean. However, do not exceed 6 strokes and be careful not to tear the sides of the groove or to allow the soil cakes to slip in the cup.

(2) Note that with some sandy and highly organic soils, it is impossible to draw the tool through the sample without tearing the sides of the groove. In such cases, the groove is made with the spatula (fig. 2-79) and shaped with the grooving tool.

(3) Before laying the grooving tool aside, clean the cutting edge by running the thumb over the tool or by washing it. This prevents the soil from drying on the tool and lost time trying to scrape or wash it clean.

d. Attach the cup to the carriage of the device and turn the crank at the rate of two revolutions per second. Count the number of blows until the two halves of the soil cake come into contact at the bottom of the groove for a distance of ½ inch (fig. 2-80). Record the number of blows.

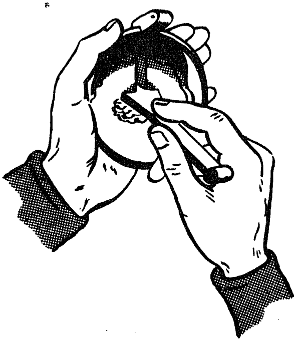


Figure 2-78. Holding cup and grooving tool.

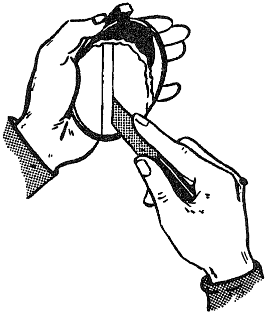


Figure 2-79. Cutting groove with spatula in sandy soil.

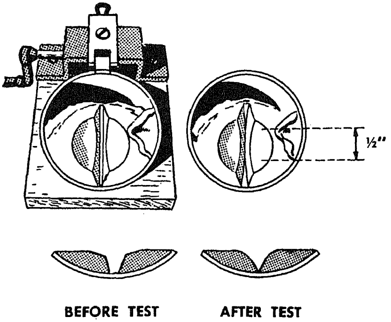


Figure 2-80. Liquid limit test.

e. Remove about 10 grams of soil from the cup by drawing the spatula from edge to edge perpendicular to the groove and through the portion of the cake that closed (fig. 2-81). This sample is used to determine the water content (para 2-54). All weighing should be accurate to ±0.01 g and the water content computed to one decimal place.

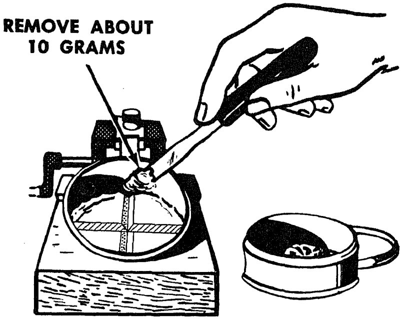


Figure 2-81. Removing moisture content portion.

f. Transfer the remaining soil in the cup to the mixing dish. Wash and dry the cup and grooving tool. Repeat steps in *b* through *e* above, for three additional samples, each of which has had the water content adjusted by drying. Drying is accomplished by continued mixing with the spatula and aided by a small electric fan, if desired. The water content adjustment must be sufficient to produce a noticeable change in the number of blows to close the groove, and results in a series with some more and some less than 25 blows which close the groove. Preferably, the range should be from 15 or more to 35 or less with two tests between 15 and 25 blows and two tests between 25 and 35 blows.

g. Material remaining in the mixing dish should be reserved for the plastic limit test.

h. Computing the liquid limit is done graphically by plotting a "flow curve" with the number of blows and the water content as coordinates (fig. 2–76). The water content, on the left edge, is plotted on an arithmetic scale and the number of blows (along the bottom) is plotted on a logarithmic scale. If the points are relatively close to lying on a straight line, a straight line is drawn as nearly as possible through the plotted points, thus establishing an average between the points. However, if the plotted points do not define a straight line, additional check runs should be made and new points plotted. The *liquid limit* is the water content corresponding to the point where the drawn straight line crosses the 25 blow coordinate. The liquid limit is read to the nearest 0.1 percent, but is reported to the nearest whole number. *Example* (fig. 2–76): In a given test, the determinations resulted in the following values:

	Number of blows	Water content
(1)	16	47. 3
(2)	23–24	46. 6
(3)	29	46. 3
(4)	33	45. 5

After plotting these four points and drawing a straight line as nearly as possible through them, the water content corresponding to the 25-blow coordinate is 46.3. Thus the liquid limit for this sample would be reported as 46.

2–59. Simplified Liquid Limit Test

The simplified test is based on the premise that the slope of the flow curve for soils within a given geologic environment is essentially constant. Thus, the liquid limit flow curve could be drawn using only one test point provided that test slope has been established by experience with other soils in the area. The simplified test should only be used in geologically similar areas and where adequate correlations have been made to define the slope of the flow curve.

a. The apparatus for the simplified test is the same as that listed in paragraph 2–57*b*. The sample is prepared as for a standard test except that its consistency is controlled to result in 20 to 30 blows to cause closure.

b. Proceed with the test as described in paragraph 2–57*a* through *d*. The amount removed from the cup for the water content determination is replaced from the prepared sample. Remix the entire amount in the cup without the addition of water. Regroove the specimen and operate the device once again. The number of blows to close the groove should be within two blows of the first measurement. If the disagreement is greater than two blows, the mixing was insufficient and the entire test must be repeated.

c. The liquid limit can be determined graphically by drawing a line through the one plotted point at the predetermined slope or by using the following equation:

$$LL=W_n\left(\frac{N}{25}\right)^{0.12}$$

Where:

W_N=water content at N blows

N=number of blows to close the groove

Values of $(N/25)^{0.12}$ are given in table 2–4. The exponent 0.12 is an average value to be used in this simplified method if the slope of the flow curve is not known for the particular soil. If the correct slope is known, the corresponding exponent is to be used. See ASTM Designation D 423 and AASHO Designation T 89.

Table 2–24. Values of $(N.25)^{0.12}$ For Liquid Limit Test

N	$(N/25)^{0.12}$
20	0. 974
21	0. 979
22	0. 985
23	0. 990
24	0. 995
25	1. 000
26	1. 005
27	1. 009
28	1. 014
29	1. 018
30	1. 022

2–60. Plastic Limit Test Preparation

a. Definition. The plastic limit of a soil is the water content, expressed as a percentage of weight of oven-dried soil, at which the soil begins to crumble when rolled into a thread ⅛ inch in diameter.

b. Apparatus. The apparatus for the plastic limit test consists of the following:

(1) Surface for rolling such a ground glass plate, a piece of linoleum, or a tabletop of close grained wood. Paper may be used as a rolling surface provided that it does not give off lint which can be picked up by the sample during rolling.

(2) Spatula.

(3) Containers (boxes, stainless steel, such as used in the liquid limit test).

(4) Balance, sensitive to 0.01 g.

(5) Oven (preferably automatically controlled for 105° to 110° C).

(6) Evaporating dish.

c. Preparation. Approximately 20 g of material is required for the plastic limit test. The sample is prepared as described for a liquid limit test or is the material remaining after the liquid limit is tested.

2–61. Plastic Limit Test Procedure

The test procedure consists of the following steps:

a. Record all identifying information for the sample on the data sheet, bottom half (fig. 2–76).

b. Take approximately 5 g of the prepared material. The sample should be taken at any stage of drying when the mass is just plastic enough to be shaped into a ball that will not stick to the fingers when squeezed.

c. Shape the test sample into a ellipsoid and roll it between the fingers and rolling surface (fig. 2–82) using enough pressure to roll the soil mass into a thread 1/8 inch in diameter. The rate of rolling should be between 80 and 90 strokes per minute. One stroke is considered a complete forward and back motion returning to the starting position.

d. When the thread diameter reaches 1/8 inch without crumbling, remold the sample into a ball and repeat the rolling process. Continue remolding and rolling until the ball has dried sufficiently for the rolled thread to crumble (fig. 2–83). The plastic limit has been reached when the thread crumbles (regardless of diameter) if the original 1/8-inch diameter has been reached at least once. Do not precipitate crumbling at exactly the 1/8-inch diameter by reducing the rate of rolling, the hand pressure, or both, and do not roll beyond the 1/8-inch diameter. It is permissible, however, to reduce the total amount of deformation for poorly plastic soils by making the initial diameter of the ellipsoidal-shaped mass nearer the required 1/8-inch final diameter. Continue to roll until all the soil has crumbled. (It does not crumble all at the same time.)

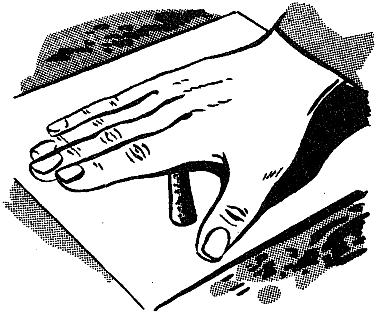


Figure 2–82. Rolling the soil, plastic limit test.

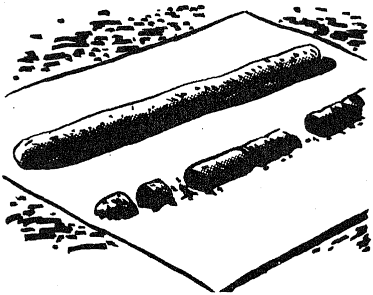


Figure 2–83. Rolled threads, uncrumbled and crumbled.

e. Collect the crumbled portions and determine the water content.

f. Repeat the process with an additional portion of the prepared sample to obtain an average value for the plastic limit. The two tests should agree within ±1 percent moisture content or the test must be repeated.

g. Record sample weight and computations on the data sheet. Weighing should be accurate to 0.01 g and water content computed to one decimal place. Report the limit to the nearest whole number.

h. Computing the plastic limit is done as follows:

Water content for each run

$$=\frac{\text{weight of water}}{\text{weight of oven-dried soil}}\times 100$$

Plastic limit=average of water content from two or more runs (fig. 2–76).

Plasticity index=liquid limit−plastic limit; or PI=LL−PL

Example (fig. 2–76): In this example, the following data were obtained. Determine the plasticity index.

Liquid limit=LL=46

Plastic limit=PL=18

Plasticity index=PI=LL−PL; PI=46−18=28

2–62. Shrinkage Factors

The shrinkage limit of a soil is the water content, expressed as a percentage of oven-dried soil, at which further loss in moisture will not cause a decrease in the volume. The shrinkage ratio, (R), and linear shrinkage, (L_s), are usually determined as part of this test. The shrinkage ratio is defined as the ratio between a given volume change and the corresponding change in water above the shrinkage limit. Linear shrinkage is the decrease in one dimension of a soil mass when the water content is reduced from a given value to the shrinkage limit. The procedure for determining shrinkage factors of soils is given in ASTM Designation D 427. Shrinkage factors are useful information for certain soils encountered in heavy frost areas (TM 5–349).

Section X. COMPACTION CONTROL TEST

2–63. Discussion

a. One of the basic construction procedures involved in building subgrades and bases for roads and airport pavements, enbankments, earth-fill dams, and similar structures is compaction. *Compaction* is the process of increasing the amounts of solids per unit volume by mechanical means. Field compaction is accomplished by rolling, or by the passage of construction equipment. Laboratory compaction may be accomplished by the impact of hammer blows, vibration, static loading, or any other method which does not alter the water content of the soil. In general, efficient and adequate compaction in the field is attained by compacting the soil at a moisture content and to a density established in the laboratory.

b. As related to compaction, the term dry "density" means dry unit weight, that is, pounds per cubic foot of soil computed on an over-dry basis. Laboratory compaction generally is accomplished by placing the soil in a cylinder of known volume and dropping a tamper of known weight onto the soil from a known height for a given number of blows. The amount of energy which is applied in this dynamic compaction procedure is called *compactive effort.* For the large majority of soils and for a given compactive effort, the density of the soil will increase to a certain point as the moisture content is increased, then start to decrease with any further increase in moisture content. The moisture content at which maximum density occurs is called the *optimum moisture content* for that compactive effort. For a given soil, each compactive effort has its own optimum moisture content; as the compactive effort is increased, the maximum density generally increases and the optimum moisture content decreases.

c. Compaction tests in the laboratory are performed to determine such properties of the soil as—

(1) The effect of varying percentages of water on dry density.

(2) The maximum density obtainable under a given compactive effort.

(3) The optimum moisture content.

d. The compaction control test used by the Corps of Engineers is the modified version of the standard AASHO* method T 180, and conforms entirely to ASTM Designation D 1557. This method is intended to determine the relationship between moisture content and density of soils when compacted in a mold of a given size with a 10 pound tamper dropped from a height of 18 inches.

2–64. Preparation

The ASTM compaction procedure may be performed either in the standard Proctor mold or the larger laboratory CBR (California bearing ratio) mold. The mold to be used as determined by particle size in the soil sample. Fine grained soils (passing No. 4 sieve) are compacted in the Proctor mold whereas soils contining up to ¾-inch material should be compacted in the CBR mold.

a. Apparatus. The apparatus used in the compaction test consists of—

(1) Compaction cylinder (Proctor mold). A cylindrical metal mold 4 inches in diameter and 4.584 inches high having a volume of 1/30 cubic foot. The mold is fitted with a detachable base plate and a removable extension collar 2½ inches high.

(2) Compaction cylinder (CBR mold). A cylindrical metal mold 6 inches in diameter and 4.5 or more inches high having a removable extension collar. If the mold is higher than 4.5 inches (usually 7 inches), the compacted sample is controlled to 4.5 or 5 inches by means of a spacer disk to act as a false bottom. The spacer disk is 1/16 inch smaller in diameter than the mold and 2 to 2½ inches high. The CBR mold is used for samples containing material retained on the No. 4 sieve and for any type of sample in the CBR test (sec. XII). The volume of the mold is 0.075 cubic foot at an inside height of 4.584 inches and 0.0818 cubic foot at an inside height of 5 inches.

(3) Compaction tamper. The tamper is a sliding weight, 2 inches in diameter and weighing 10 pounds, which is attached to a plunger and handle that move with the tamper in a tamper guide (fig. 2–84). The height of fall is designed to be 18 inches.

(4) Balances, 25 kg capacity, sensitive to 1 g and 5 kg capacity, sensitive to 0.1 g.

(5) Three sieves to process the soil (2 inch, ¾ inch, and No. 4).

(6) Straightedge, steel.

(7) Mixing tools, such as pans, spoon, trowel, spatula, or electric mixer.

(8) Other general laboratory equipment including that necessary to perform moisture content determinations (para. 2–54).

b. Sample Preparation. Samples are prepared dif-

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*American Association of State Highway Officials.

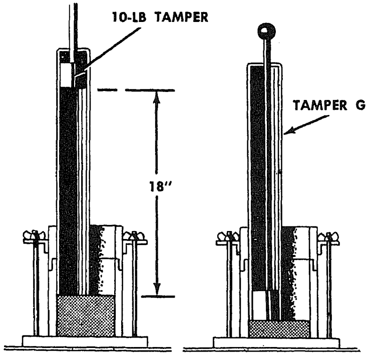


Figure 2–84. Soil compaction in a mold.

ferently depending on whether they are fine grained or gravelly soils.

(1) Fine grained soils with no material retained on the No. 4 sieve require approximately 30 pounds, dry weight, of soil (about 5 pounds for each mold) for the test. The soil is air-dried until it can pass readily through the No. 4 sieve. All lumps are broken up into individual particles to obtain a uniform mixture. After screening through No. 4 sieve, the sample is thoroughly mixed and an initial moisture-content determination is made. The sample is stored in an airtight container until ready for use.

(2) Gravelly soils are compacted in the CBR mold using material which passes the ¾-inch sieve. The sample is air-dried and screened over a ¾-inch sieve. If any particles are larger than ¾ inch, a processing-for-size correction is required before the test. Approximately 75 pounds, dry weight, of processed soil is required for the test (12 pounds per mold). To determine the size corrections, separate the material on the ¾-inch and the No. 4 sieves. Weigh the three fractions and compute the *percentage by weight* retained on the two sieves and passing the No. 4 sieve. The material retained on the ¾-inch sieve is to be discarded. It is then, in effect, replaced in the sample from the portion retained on the No. 4 sieve. One method of doing this is to sieve an additional sample of the material and use the portion retained on the No. 4 sieve to replace the discarded material greater than ¾-inch of the total soil sample. However, once the desired percentages are computed, the amount retained on No. 4 could remain constant and the portion passing No. 4 reduced to correct proportions as indicated in the example below. The reconstituted material and the portion passing the No. 4 sieve are recombined in the desired proportions, well mixed, and stored in an airtight container until ready for use. Reconstituting the soil restricts the maximum size of particles and avoids the problem of trying to compact material into a layer thinner than the largest size particles.

(3) Example (size correction for gravelly soils). Total weight of an air-dried natural sample is 110 pounds. The table gives the results of soil compaction test.

	Natural sample		*Processed sample*	
Screen size	*Retained on screen lb*	*Retained on screen percent*	*Desired percentage percent*	*Computed weight lb*
¾ inch	16. 0	14. 6	0	0
No. 4	44. 0	40. 0	54. 6	44. 0
Passing No. 4	50. 0	45. 4	45. 4	36. 5
Total	110. 0	100. 0	100. 0	80. 5

This computation permits using all of the material retained on the No. 4 sieve, and making the correction by reducing some of the material which has passed the No. 4 sieve. If the weight of the material passing the sieve was held constant, more natural material would have to be sieved while the original 44 pounds retained on the No. 4 sieve would have to be transferred and held during this extra step.

2–65. Procedure

a. The compaction test procedures for the fine grained soils and the gravelly soils are the same with two exceptions: first, the Proctor mold is used for the fine and the CBR mold for the gravelly, and second, 25 tamper blows per layer are used in the Proctor mold and 55 blows per layer in the CBR mold. This results in equal compactive efforts for the two molds and the different sizes of soil. The test procedure is as follows:

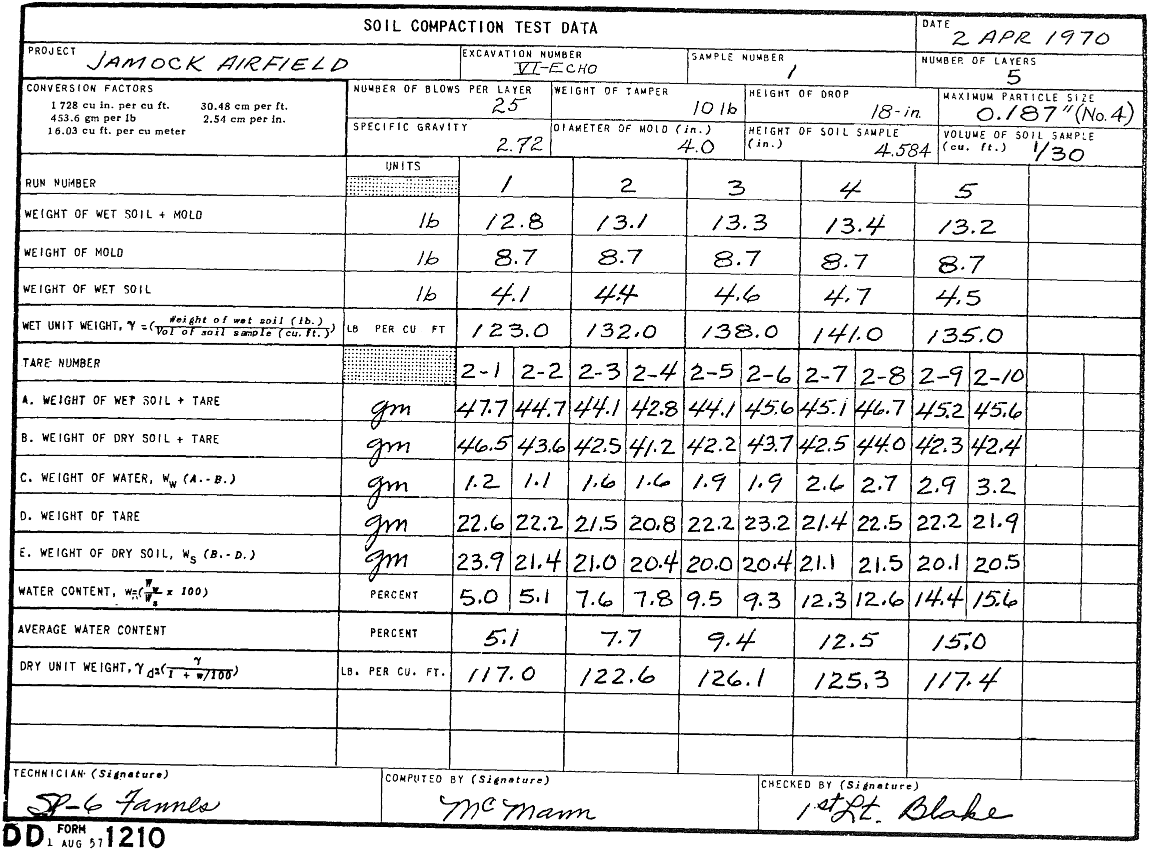
(1) From the prepared material (para 2–64), weigh out about 12–15 pounds (CBR mold) or 5–7 pounds (Proctor mold) dry weight of soil.

(2) Determine the quantity of water which, when mixed with the soil, will cause it to adhere or ball together slightly when squeezed firmly in the palm of the hand. Add the water to the sample and mix thoroughly.

(3) Store the water-mixed material in an airtight container for approximately 24 hours to permit the moisture content to become uniform.

(4) Weigh out at least four additional specimens ((1) above). For each specimen, progressively increase the water content (determined in (2) above) about 2 percent over the previous one. (For adjustment of water content, see para 2–66). Mix and store each specimen in a separate airtight container. Note that sandy soils absorb moisture readily and 24 hour storage may not be required. In this case, separate the five or more specimens and add the water before testing. *Do not reuse a tested specimen.*

(5) Cover the proper mold with a light coat of oil and weigh it to the nearest gram. Do not include base plate and extension collar. Record the weight on the data sheet, DD Form 1210 (fig. 2–85).



SOIL COMPACTION TEST DATA

PROJECT JAMOCK AIRFIELD	EXCAVATION NUMBER VI-ECHO	SAMPLE NUMBER 1		DATE 2 APR 1970 NUMBER OF LAYERS 5
CONVERSION FACTORS 1 728 cu in. per cu ft. 453.6 gm per lb 16.03 cu ft. per cu meter 30.48 cm per ft. 2.54 cm per in.	NUMBER OF BLOWS PER LAYER 25	WEIGHT OF TAMPER 10 lb	HEIGHT OF DROP 18-in.	MAXIMUM PARTICLE SIZE 0.187"(No.4)
	SPECIFIC GRAVITY 2.72	DIAMETER OF MOLD (in.) 4.0	HEIGHT OF SOIL SAMPLE (in.) 4.584	VOLUME OF SOIL SAMPLE (cu. ft.) 1/30

	UNITS											
RUN NUMBER		1		2		3		4		5		
WEIGHT OF WET SOIL + MOLD	lb	12.8		13.1		13.3		13.4		13.2		
WEIGHT OF MOLD	lb	8.7		8.7		8.7		8.7		8.7		
WEIGHT OF WET SOIL	lb	4.1		4.4		4.6		4.7		4.5		
WET UNIT WEIGHT, $\gamma = \left(\frac{\text{Weight of wet soil (lb.)}}{\text{Vol of soil sample (cu. ft.)}}\right)$	LB PER CU FT	123.0		132.0		138.0		141.0		135.0		
TARE NUMBER		2-1	2-2	2-3	2-4	2-5	2-6	2-7	2-8	2-9	2-10	
A. WEIGHT OF WET SOIL + TARE	gm	47.7	44.7	44.1	42.8	44.1	45.6	45.1	46.7	45.2	45.6	
B. WEIGHT OF DRY SOIL + TARE	gm	46.5	43.6	42.5	41.2	42.2	43.7	42.5	44.0	42.3	42.4	
C. WEIGHT OF WATER, W_W (A.-B.)	gm	1.2	1.1	1.6	1.6	1.9	1.9	2.6	2.7	2.9	3.2	
D. WEIGHT OF TARE	gm	22.6	22.2	21.5	20.8	22.2	23.2	21.4	22.5	22.2	21.9	
E. WEIGHT OF DRY SOIL, W_S (B.-D.)	gm	23.9	21.4	21.0	20.4	20.0	20.4	21.1	21.5	20.1	20.5	
WATER CONTENT, $w = \left(\frac{W_w}{W_s} \times 100\right)$	PERCENT	5.0	5.1	7.6	7.8	9.5	9.3	12.3	12.6	14.4	15.6	
AVERAGE WATER CONTENT	PERCENT	5.1		7.7		9.4		12.5		15.0		
DRY UNIT WEIGHT, $\gamma_d = \left(\frac{\gamma}{1 + w/100}\right)$	LB. PER CU. FT.	117.0		122.6		126.1		125.3		117.4		

TECHNICIAN (Signature) SP-6 Fannes	COMPUTED BY (Signature) McMann	CHECKED BY (Signature) 1st Lt. Blake

DD FORM 1 AUG 57 **1210**

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Figure 2-85. Sample data sheet, compaction control test (DD form 1210).

(6) Prepare the mold by attaching the collar and mounting the base, if necessary, and setting it on a concrete floor or other rigid support, such as a pedestal. If using the CBR mold with the spacer disk, place filter paper on top of the disk.

(7) Place a sufficient amount of the first prepared specimen into the mold to result in a 1-inch compacted layer (usually between 1- and 1½-inch uncompacted thickness).

(8) Compact the material in the mold with 25 blows (55 blows in the CBR mold) of the 10-pound tamper from a height of 18 inches (fig. 2–84). Hold the tamper vertically, place its face on top of the soil, raise the handle until it reaches the top, and let the weight fall freely onto the soil. Change the position of the guide and tamper, and repeat the process until the soil layer has received the prescribed number of blows. Be sure that the tamping is evenly distributed around the soil in the mold. After compacting each layer, check the height of the compacted layer to see if the compacted thickness is 1 inch; if more than 1 inch in thickness, add less soil for the next layer; if less than 1 inch, add more soil for the next layer.

(9) Using a sharp pointed instrument, scarify the surface of the compacted layer to a depth of about ¼ inch; a grid of scratch lines ¾ inch to 1 inch apart provides the proper degree of scarifying. Add the next layer of the speciment ((7) above) and repeat the tamping process ((8) above). Continue adding and tamping until five layers have been compacted in the mold. The fifth layer should extend slightly above the top of the mold, and into the collar.

(10) With a knife or spatula, cut around the inside edge of the collar to prevent shearing the compacted soil when removing the collar. Carefully remove the collar and using the straightedge or spatula, trim the excess portion above the exact level of the top of the mold (fig. 2–86). Use a sawing back and forth motion of the straightedge to trim the excess soil; start at the center of the mold and work outward, first to one side and then to the other.

(11) Remove mold, with the compacted specimen in it, from the base plate and weigh mold plus specimen. Record weight of mold plus soil (fig. 2–85).

(12) Take two samples of the compacted soil, one from the top and one from the bottom (fig. 2–87), for a moisture-content determination. Weigh these samples to 0.1 g and save them for the remaining moisture-content procedures.

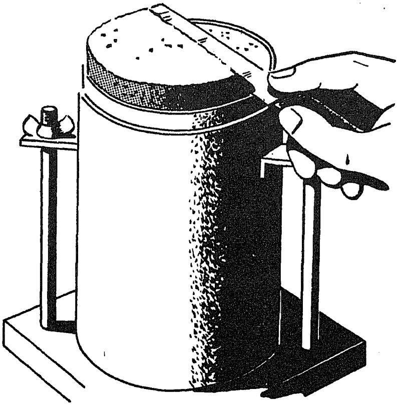


Figure 2–86. Trimming the compacted soil sample.

NOTE

(Take sample for moisture content from loose layer before compacting if sample is used for CBR determination.)

(13) Repeat (6) through (12) above, for the remaining specimens.

(14) After completing the compaction portion of the test, complete the moisture-content determinations (para 2–54).

b. The following computations can be made with data obtained:

(1) *Wet unit weight.* The weight of the compacted specimen and the mold (*a*(11) above) minus the weight of the mold alone (*a*(5) above) is the weight of the compacted specimen. The wet unit weight is equal to the weight of the compacted specimen divided by the volume of the mold. The volume of the Proctor mold is $\frac{1}{30}$ cubic foot; the 7-inch CBR mold with 2-inch spacer is 0.0818 cubic foot and with a 2½-inch spacer is 0.0750 cubic foot.

(2) *Moisture content* (*w*). This is determined as described in paragraph 2–54,

$$w=\frac{\text{wt of water}}{\text{wt of dry solids}}\times 100.$$

(3) *Dry unit weight.*

$$\text{Dry unit weight} = \frac{\text{wet unit weight}}{1 + \frac{\text{moisture content}}{100}}$$

(4) *Optimum moisture content (OMC).* The OMC is determined from a curve obtained by plotting the moisture content against the equivalent dry unit weight on arithmetic graph paper. The plotted points are joined to form a curve (DD Form 1211 (fig. 2–88)). The highest point on the curve is designated 100 percent or maximum modified AASHO density and the moisture content corresponding to this reading is the optimum moisture content for this density. In figure 2–88, the maximum density for the 55-blow curve is 127.2 pounds per cubic foot and optimum moisture content is 10.9 percent.

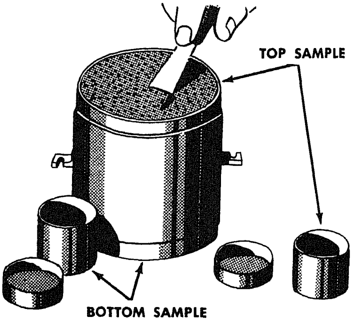
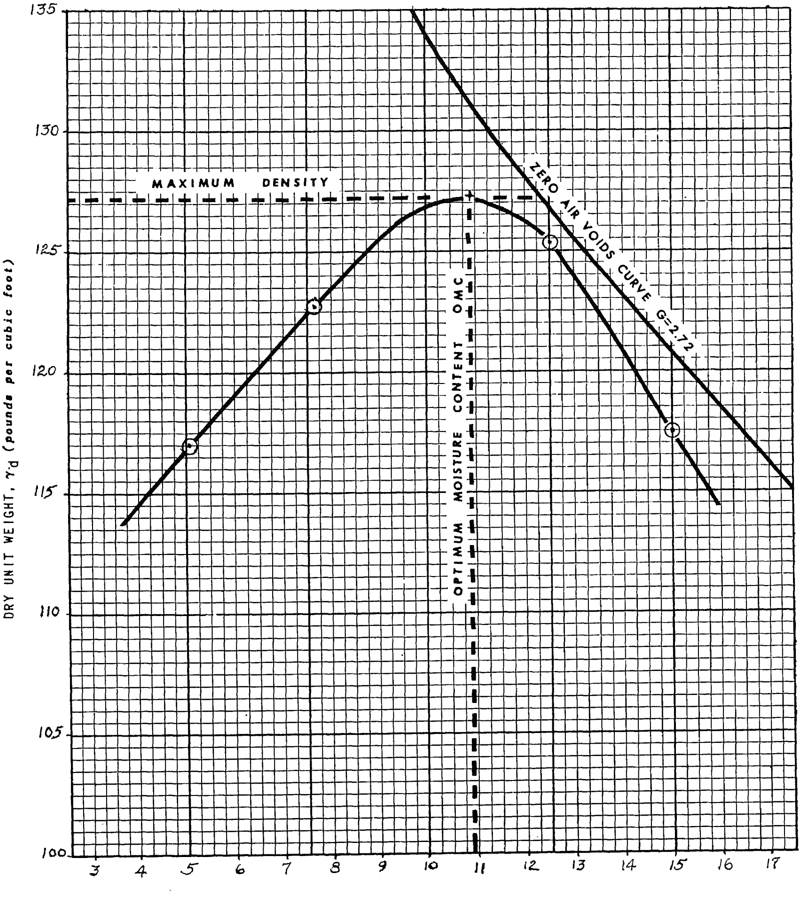


Figure 2–87. Taking moisture-content samples of a compacted specimen.

SOIL COMPACTION TEST GRAPH	DATE 2 APR 1970

PROJECT JAMOCK AIRFIELD		RESULTS
EXCAVATION NUMBER VI-ECHO	SAMPLE NUMBER 1	MAXIMUM DRY UNIT WEIGHT 127.2 lb/cu ft
REMARKS *(Use reverse side, if more space is needed)* FINE-GRAINED SOIL.		OPTIMUM MOISTURE *(Water)* CONTENT 10.9 %



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DD FORM 1 AUG 57 **1211**

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Figure 2–88. Determination of optimum moisture content.

2–66. Adjustment of Moisture Content

a. The OMC curve is drawn with dry density and moisture content as coordinates. To construct an acceptable curve, at least two of the plotted points should fall on one side and two on the other side of the OMC. However, it is important to remember that during the testing period the only density that can be determined is the wet soil density or wet unit weight. To compute the dry density, the moisture content must be determined and this can take up to 24 hours.

b. An acceptable curve can be obtained by starting at a water content to produce a sample just wet enough to ball together when squeezed and released. Additional points can be obtained by increasing the water content by 2 percent for each of four subsequent samples. This may not always be true. At times, it may be necessary to increase the water by as much as 4 percent per step to get the proper curve.

c. One method to determine when the OMC has been reached while testing is to compare the wet weights of the various samples after compaction. The OMC and maximum density have been passed when the wetter compacted sample shows a decline in weight over the previous drier sample. If this occurs with the third or fourth sample, the curve will have at least two plotted points on either side of the OMC.

d. To aid in the selection of increments (2 percent to 4 percent increases) hints can be taken from typical curves. Generally, the more plastic the soil, the flatter the curve, and the higher the OMC. By experience, it is possible to select appropriate increments for the soil to be tested.

e. To compute the amount of water required to raise the moisture content of a soil to a certain percentage, the following data must be known:

Weight of soil sample
Present moisture content
Desired moisture content

An example of the computation is given below: Assume a 12 pound sample with a 3 percent moisture content must be raised to 10 percent.

$$\text{Dry weight} = \frac{\text{wet weight}}{1+\frac{w}{100}} = \frac{12}{1+\frac{3}{100}} = \frac{12}{1.03} = 11.65 \text{ lb}$$

where moisture content (w) is—

$$w = \frac{\text{weight of water}}{\text{weight of dry soil}} \times 100$$

hus: Weight of water

$$= \frac{\text{moisture content} \times \text{dry weight of soil}}{100}$$

$$= \frac{7 \times 11.65}{100} = 0.816 \text{ pound}$$

The 7 percent moisture content is derived from the required value (10 percent) minus the content in the soil (3 percent).

To convert to grams:

$$0.816 \text{ pound} \times 454 \text{g/lb} = 370 \text{ g.}$$

The formulas above may be combined into one formula, thus: Water to be added (in grams, cc, or ml)

$$\left(\frac{\text{wet soil weight (lb)}}{1+\frac{\text{moisture content}}{100}}\right) \times 454 \times \frac{\text{moisture difference (\%)}}{100}$$

2–67. Degree of Saturation

a. At complete saturation, the voids (space between particles) in the soil mass are completely filled with water, that is, no air is present, and the degree of saturation, (S), is equal to 100 percent. The zero air voids curve (100 percent saturation) for the soil used (specific gravity, G, equal to 2.72) is shown on the plot of figure 2–88. This curve is obtained by plotting dry densities corresponding to complete saturation at different moisture contents.

b. When saturation is complete, (S=100), the moisture content, (w_{100}), required to fill all voids is dependent upon only the specific gravity of the soil particles (G) and the dry unit weight, (w_d),

$$w_{(100)} = \frac{62.43}{w_d} - \frac{1}{G}$$

(The unit weight of 1 cubic foot of water is 62.4 lb). This expression may be used to compute points for plotting the zero air voids curve. Table 2–5 gives values of water content and densities at zero air voids (complete saturation) for the usual range of specific gravity and dry density. The specific gravity of the soil is determined by the described methods.

c. The zero air voids curve represents theoretical values which are practically unattainable, because it is not possible to remove all the air contained in the voids of a soil by compaction alone. Typically, at moisture contents beyond optimum, the actual compaction curve closely parallels theoretically perfect compaction curve. Any values of dry density which plot to the right of the zero air voids curve are in error. The error may be in the test measurement, the calculations, or the specific gravity.

d. It is sometimes desirable to know the degree of saturation corresponding to points on the actual compaction curve. With the plot of figure 2–88 available, the degree of saturation which corresponds to any point on the moisture-density curve may be calculated by dividing the actaul water content of the specimen by the water content which corresponds to 100 percent saturation (at the same density).

Table 2-5. Data for Zero Air Voids Curve

Specific gravity of soil	Water content, w_{100}, in percent of dry weight for dry unit weight, w_d, in pounds per cubic foot of 50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130	135	140	145
2.40	83.2	71.8	62.4	54.4	47.5	41.6	36.4	31.8	27.7	24.1	20.8	17.8	15.1	12.6	10.4	8.3	6.4	4.6	2.9	1.4
2.45	84.0	72.7	63.2	55.2	48.4	42.4	37.2	32.6	28.6	24.9	21.6	18.6	15.9	13.5	11.2	9.1	7.2	5.4	3.8	2.2
2.50	84.9	73.5	64.1	56.1	49.2	43.2	38.0	33.5	29.4	25.7	22.4	19.5	16.8	14.3	12.0	9.9	8.0	6.2	4.6	3.1
2.51	85.0	73.7	64.2	56.2	49.4	43.4	38.2	33.6	29.5	25.9	22.6	19.6	16.9	14.5	12.2	10.1	8.2	6.4	4.8	3.2
2.52	85.2	73.8	64.4	56.4	49.5	43.6	38.4	33.8	29.7	26.0	22.8	19.8	17.1	14.6	12.4	10.3	8.3	6.6	4.9	3.4
2.53	85.3	74.0	64.5	56.5	49.7	43.7	38.5	33.9	29.8	26.2	22.9	19.9	17.2	14.8	12.5	10.4	8.5	6.7	5.1	3.5
2.54	85.5	74.1	64.7	56.7	49.8	43.9	38.7	34.1	30.0	26.4	23.1	20.1	17.4	14.9	12.7	10.6	8.7	6.9	5.2	3.7
2.55	85.6	74.3	64.8	56.8	50.0	44.0	38.8	34.2	30.2	26.5	23.2	20.2	17.5	15.1	12.8	10.7	8.8	7.0	5.4	3.8
2.56	85.8	74.5	65.0	57.0	50.1	44.2	39.0	34.4	30.3	26.7	23.4	20.4	17.7	15.2	13.0	10.9	9.0	7.2	5.5	4.0
2.57	86.0	74.6	65.1	57.1	50.3	44.3	39.1	34.5	30.5	26.8	23.5	20.6	17.8	15.4	13.1	11.0	9.1	7.3	5.7	4.2
2.58	86.1	74.8	65.3	57.3	50.4	44.5	39.3	34.7	30.6	27.0	23.7	20.7	18.0	15.5	13.3	11.2	9.3	7.5	5.8	4.3
2.59	86.3	74.9	65.4	57.4	50.6	44.6	39.4	34.8	30.8	27.1	23.8	20.9	18.1	15.7	13.4	11.3	9.4	7.6	6.0	4.5
2.60	86.4	75.1	65.6	57.6	50.7	44.8	39.6	35.0	30.9	27.3	24.0	21.0	18.3	15.8	13.6	11.5	9.6	7.8	6.1	4.6
2.61	86.6	75.2	65.7	57.7	50.9	44.9	39.7	35.1	31.1	27.4	24.1	21.2	18.4	16.0	13.7	11.6	9.7	7.9	6.3	4.8
2.62	86.7	75.3	65.9	57.9	51.0	45.1	39.9	35.3	31.2	27.6	24.3	21.3	18.6	16.1	13.9	11.8	9.9	8.1	6.4	4.9
2.63	86.8	75.5	66.0	58.0	51.2	45.2	40.0	35.4	31.4	27.7	24.4	21.4	18.7	16.3	14.0	11.9	10.0	8.2	6.6	5.0
2.64	87.0	75.6	66.2	58.2	51.3	45.4	40.2	35.6	31.5	27.8	24.6	21.6	18.9	16.4	14.2	12.1	10.1	8.4	6.7	5.2
2.65	87.1	75.8	66.3	58.3	51.5	45.5	40.3	35.7	31.6	28.0	24.7	21.7	19.0	16.6	14.3	12.2	10.3	8.5	6.9	5.3
2.66	87.3	75.9	66.5	58.5	51.6	45.7	40.5	35.9	31.8	28.1	24.8	21.9	19.2	16.7	14.4	12.4	10.4	8.7	7.0	5.5
2.67	87.4	76.1	66.6	58.6	51.7	45.8	40.6	36.0	31.9	28.3	25.0	22.0	19.3	16.8	14.6	12.5	10.6	8.8	7.1	5.6
2.68	87.6	76.2	66.7	58.7	51.9	45.9	40.7	36.1	32.1	28.4	25.1	22.2	19.4	17.0	14.7	12.6	10.7	8.9	7.3	5.8
2.69	87.7	76.3	66.9	58.9	52.0	46.1	40.9	36.3	32.2	28.6	25.3	22.3	19.6	17.1	14.9	12.8	10.9	9.1	7.4	5.9
2.70	87.8	76.5	67.0	59.0	52.2	46.2	41.0	36.4	32.3	28.7	25.4	22.4	19.7	17.3	15.0	12.9	11.0	9.2	7.6	6.0
2.71	88.0	76.6	67.2	59.2	52.3	46.3	41.1	36.6	32.5	28.8	25.5	22.6	19.9	17.4	15.1	13.0	11.1	9.3	7.7	6.2
2.72	88.1	76.8	67.3	59.3	52.4	46.5	41.3	36.7	32.6	29.0	25.7	22.7	20.0	17.5	15.3	13.2	11.3	9.5	7.8	6.3
2.73	88.2	76.9	67.4	59.4	52.6	46.6	41.4	36.8	32.7	29.1	25.8	22.8	20.1	17.7	15.4	13.3	11.4	9.6	8.0	6.4
2.74	88.4	77.0	67.6	59.6	52.7	46.7	41.5	37.0	32.9	29.2	25.9	23.0	20.3	17.8	15.5	13.4	11.5	9.7	8.1	6.6
2.75	88.5	77.2	67.7	59.7	52.8	46.9	41.7	37.1	33.0	29.4	26.1	23.1	20.4	17.9	15.7	13.6	11.7	9.9	8.2	6.7
2.76	88.6	77.3	67.8	59.8	53.0	47.0	41.8	37.2	33.1	29.5	26.2	23.2	20.5	18.1	15.8	13.7	11.8	10.0	8.4	6.8
2.77	88.8	77.4	68.0	60.0	53.1	47.1	41.9	37.4	33.3	29.6	26.3	23.4	20.7	18.2	15.9	13.8	11.9	10.1	8.5	7.0
2.78	88.9	77.5	68.1	60.1	53.2	47.3	42.1	37.5	33.4	29.8	26.5	23.5	20.8	18.3	16.1	14.0	12.1	10.3	8.6	7.1
2.79	89.0	77.7	68.2	60.2	53.4	47.4	42.2	37.6	33.5	29.9	26.6	23.6	20.9	18.5	16.2	14.1	12.2	10.4	8.8	7.2
2.80	89.2	77.8	68.3	60.3	53.5	47.5	42.3	37.7	33.7	30.0	26.7	23.8	21.0	18.6	16.3	14.2	12.3	10.5	8.9	7.4
2.81	89.3	77.9	68.5	60.5	53.6	47.7	42.5	37.9	33.8	30.1	26.8	23.9	21.2	18.7	16.4	14.4	12.4	10.7	9.0	7.5
2.82	89.4	78.1	68.6	60.6	53.7	47.8	42.6	38.0	33.9	30.3	27.0	24.0	21.3	18.8	16.6	14.5	12.6	10.8	9.1	7.6
2.83	89.5	78.2	68.7	60.7	53.9	47.9	42.7	38.1	34.0	30.4	27.1	24.1	21.4	19.0	16.7	14.6	12.7	10.9	9.3	7.7
2.84	89.7	78.3	68.8	60.8	54.0	48.0	42.8	38.2	34.2	30.5	27.2	24.3	21.5	19.1	16.8	14.7	12.8	11.0	9.4	7.9
2.85	89.8	78.4	69.0	61.0	54.1	48.2	43.0	38.4	34.3	30.6	27.3	24.4	21.7	19.2	16.9	14.9	12.9	11.2	9.5	8.0
2.86	89.9	78.5	69.1	61.1	54.2	48.3	43.1	38.5	34.4	30.8	27.5	24.5	21.8	19.3	17.1	15.0	13.1	11.3	9.6	8.1
2.87	90.0	78.6	69.2	61.2	54.4	48.4	43.2	38.6	34.5	30.9	27.6	24.6	21.9	19.5	17.2	15.1	13.2	11.4	9.8	8.2
2.88	90.1	78.8	69.3	61.3	54.5	48.5	43.3	38.7	34.7	31.0	27.7	24.7	22.0	19.6	17.3	15.2	13.3	11.5	9.9	8.3
2.89	90.3	78.9	69.5	61.5	54.6	48.6	43.4	38.9	34.8	31.1	27.8	24.9	22.2	19.7	17.4	15.3	13.4	11.6	10.0	8.5

Table 2-5. Data for Zero Air Voids Curve — Continued

2.90	90.4	79.0	69.6	61.6	54.7	48.8	43.6	39.0	34.9	31.2	28.0	25.0	22.3	19.8	17.6	15.5	13.5	11.8	10.1	8.6
2.95	91.0	79.6	70.2	62.2	55.3	49.3	44.1	39.6	35.5	31.8	28.5	25.6	22.9	20.4	18.1	16.0	14.1	12.3	10.7	9.2
3.00	91.5	80.2	70.7	62.7	55.9	49.9	44.7	40.1	36.0	32.4	29.1	26.1	23.4	21.0	18.7	16.6	14.7	12.9	11.3	9.7
3.05	92.1	80.7	71.3	63.3	56.4	50.5	45.3	40.7	36.6	32.9	29.6	26.7	24.0	21.5	19.2	17.2	15.2	13.5	11.8	10.3
3.15	93.1	81.8	72.3	64.3	57.4	51.5	46.3	41.7	37.6	34.0	30.7	27.7	25.0	22.5	20.3	18.2	16.3	14.5	12.8	11.3

Note: Zero air voids curve equivalent to a degree of saturation, S, equal to 100 per cent.
Data for zero aid voids curve computed from the formula in paragragh 2-67b.

For example: at maximum density for the soil represented by the plot of figure 2–88, the actual water content is 10.9 percent; at 100 percent saturation and the same density, the corresponding moisture is 12.3 percent. Therefore, the degree of saturation and maximum density is equal to 10.9% /12.3% = 0.89 or 89%.

2–68. Other Compactive Efforts

Under some circumstances, it may be necessary to use a compactive effort in the laboratory which is different from the modified AASHO (or ASTM) compaction procedure. The usual reason for doing this is to study the effect of variation in density upon some property of the soil, such as the CBR. In this case, samples are compacted using the procedures previously described, except that the variation in density is achieved by varying the number of blows applied to each layer and by choosing one of the two tampers available. In unusual circumstances, the laboratory compaction procedure may be changed to produce a compactive effort which more closely resembles the energy that can be put into the soil by the use of available rolling equipment. In the CBR test, the compaction procedure calls for 12, 26, or 55 blows per layer. This is discussed under the California Bearing Ratio (sec XII).

Section XI. DENSITY DETERMINATION

2–69. Introduction

Proper field control is of paramount importance in earthwork construction. The control tests are conducted on the soil at the jobsite as construction proceeds. If at any time the tests indicate that operations are not producing a soil condition specified by the design tests, immediate action should be taken to remedy the situation. Additional facts about the control of earthwork construction are contained in TM 5–335.

a. The stress-deformation characteristics of a soil are directly related to the moisture content and density of the soil. Consequently, it is possible to set up the specifications for a given soil on the basis of moisture content and density, as indicated in paragraph 2–63. Field control then consists of taking samples for moisture content and density determinations as construction proceeds. Densities obtained are compared with minimum densities established for the particular job, while water contents are compared, generally, with the optimum moisture content previously established to see that compaction is taking place within the desired range or to permit its adjustment, if necessary.

b. An undisturbed sample of known or measurable dimensions provides information for the computation of density and the moisture content of the soil. However, if the soil is not in proper condition during construction to remove an undisturbed sample, the density determination requires measuring the volume of the hole after the sample is removed. The procedure consists of filling the hole with a measured quantity of a known-density material, such as sand, oil, or water, and computing the volume of the hole which is equal to the volume of soil removed. The soil's moisture content and density are then determined.

c. The method for in-place density depends upon the type of soil encountered and the equipment available. On moist, cohesive, fine grained soils, undisturbed samples (para 2–17) using the samplers may be sufficient. Coarse grained soils or cohesionless soils make it difficult to obtain an undisturbed sample. In these soils, density determination by the displacement method may be required. Sand-displacement may be used on any type of base course or subgrade material. Oil-displacement cannot be used on highly pervious soils, or crushed stone or slag base courses. In addition, if the pavement to be used is asphaltic concrete, the residual oil and spillage will tend to soften the asphalt. Water-displacement requires the use of a balloon to contain the water and can be used on any type of soil.

2–70. Sand-Displacement Equipment Preparation

The sand-displacement method may be used in either fine or coarse grained materials. This method is so named because a calibrated sand is used to determine the volume of the hole from which a sample has been taken.

a. Requirements. The test consists of digging out a sample of the material to be tested, determining the volume of the hole, and determining the dry weight of the sample. There are three requirements that must be met for this test.

(1) The volume of the sample must be 0.05 cubic foot or larger. Samples of 0.05-cubic foot volume are satisfactory for materials with a maximum aggregate size of about ½ inch. For 1-inch particles a minimum of 0.075 cubic foot, and for 2-inch particles 0.100 cubit foot are required.

(2) A double-cone cylinder must be used. This permits calibrating the sand for each test performed.

(3) The sand must be clean, dry, and free flowing with a constant moisture content while performing the test. Uniformly-graded and well-rounded sand passing a No. 20 sieve and retained on a No. 40 sieve is most suitable for this test. There should be practically no material finer than the No. 200 sieve in the sand.

b. Equipment. The essential equipment, some of which is shown in figure 2–89, consists of—

Can, friction-top, 1-gallon capacity
Cylinder, soil-compaction
Digger, density
Hammer
Knife
Nails, 20 d, common
Pie plate
Scale, bench, twin weigh beams, 21,100-gram capacity
Spoon, cooking
Straightedge, steel
Tester, soil density, sand method, double cone assembly, with 1-gallon sand jug
Tray, template, soil density, 4-in. hole

A 4-in. posthole auger, a bake pan, and moisture content cans are also needed. A paint brush with moderately long bristles and some modeling clay will be needed in some cases.

c. Calibration. The volume of the jar and connecting cone up through the valve, and the empty weight of the apparatus must be known before the sand can be calibrated. A data sheet (fig. 2–90) should be prepared for recording the information. The sand calibration is repeated prior to each test. One other calibration must be made to account for surface irregularities between the surface of the cone base and the surface to be tested.

(1) *Apparatus calibration.* To determine the weight and volume of the apparatus, the procedure is—

(*a*) Weigh the empty dry cone and jar, and record the weight (in grams).

(*b*) Screw the cone tightly on the jar, place apparatus upright and open valve.

(*c*) Fill the apparatus with water at room temperature until the water level stands some distance up in the top cone. Make sure no air bubbles are trapped in the apparatus.

(*d*) Close the cone valve, pour off the excess water, shake the apparatus to check for entrapped bubbles, and dry the cone and outside of the apparatus.

(*e*) Weigh the water-filled apparatus and record the weight (in grams).

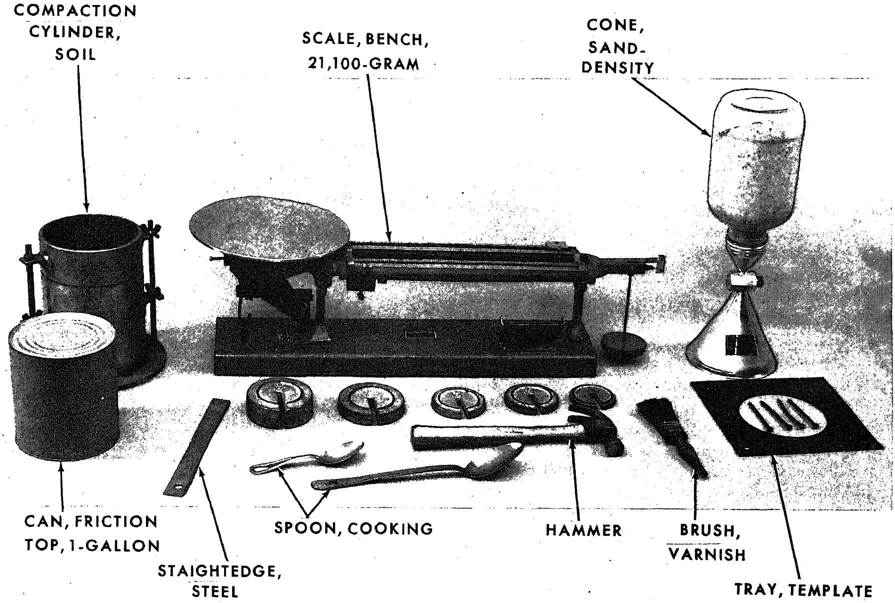


Figure 2–89. Sand-displacement method apparatus.

(*f*) Empty the water from the apparatus.

(*g*) Repeat (*b*) through (*f*) above, at least two or more times.

(*h*) Compute the volume of the apparatus by averaging the measured weights, converting the average to pounds (divide by 453.6 g/lb), and converting the average weight in pounds to volume (divide by 62.4 pounds per cubic foot). The result is the volume in cubic feet. Note the example of the computation in figure 2-90.

(2) *Sand calibration.* The bulk density of sand is affected by changes in temperature and humidity. Sand calibration must be checked prior to each test. The information is recorded on DD Form 1215 (fig. 2-91). The procedure is as follows:

(*a*) Weigh the empty cone and jar, and record the weight (line 2, fig. 2-91).

(*b*) Check the cone to see that it is screwed tightly on the jar and place the apparatus on a firm, level surface.

(*c*) Close the cone valve and fill the upper cone with air-dried sand.

(*d*) Open the cone valve and as the sand runs into the jar, keep adding more sand to the upper cone to maintain a steady, full stream of sand through the valve. Be careful not to jar the apparatus while the sand is being poured.

(*e*) When the jar and lower cone are filled, and the sand stops flowing, close the valve.

(*f*) Dump the excess sand from the upper cone.

(*g*) Weigh the sand-filled apparatus and record the weight (line 1, fig. 2-91).

(*h*) Compute the calibrated density (in lb/cu ft) by subtracting the weight of the empty apparatus from the weight of the filled apparatus (line 1 minus 2 equals line 3, fig. 2-91), and dividing first by 453.6 to convert grams to pounds, and then by the volume of the apparatus (line 4).

(*i*) An alternate method of sand calibration is to use a container of known volume, such as the Proctor mold. The mold with base plate attached is weighed empty. The collar is attached and the sand cone with the valve open is placed on top. The sand is poured from about the same height that it will fall in a field-dug hole and allowed to fall at its own rate into the mold. When the sand stops running into the mold, the valve on the cone is closed. The sand cone and the mold collar are carefully removed, in turn. The excess sand remaining on top of the mold is struck off with a straightedge. Any sand adhering to the outside of the mold is brushed off. The mold, full of sand, is weighed. The entire process is repeated and the two weights (mold full of sand) are averaged. The weight of the empty mold is subtracted from the average of the full weights. The difference in weights divided by the known volume of the container is equal to the density of the sand.

(3) *Surface calibration.* One more calibration must be made prior to starting the test. However, it is performed after the surface is prepared and the template tray is in place (para 2-71*a*(2)).

CALIBRATION OF SAND-CONE APPARATUS

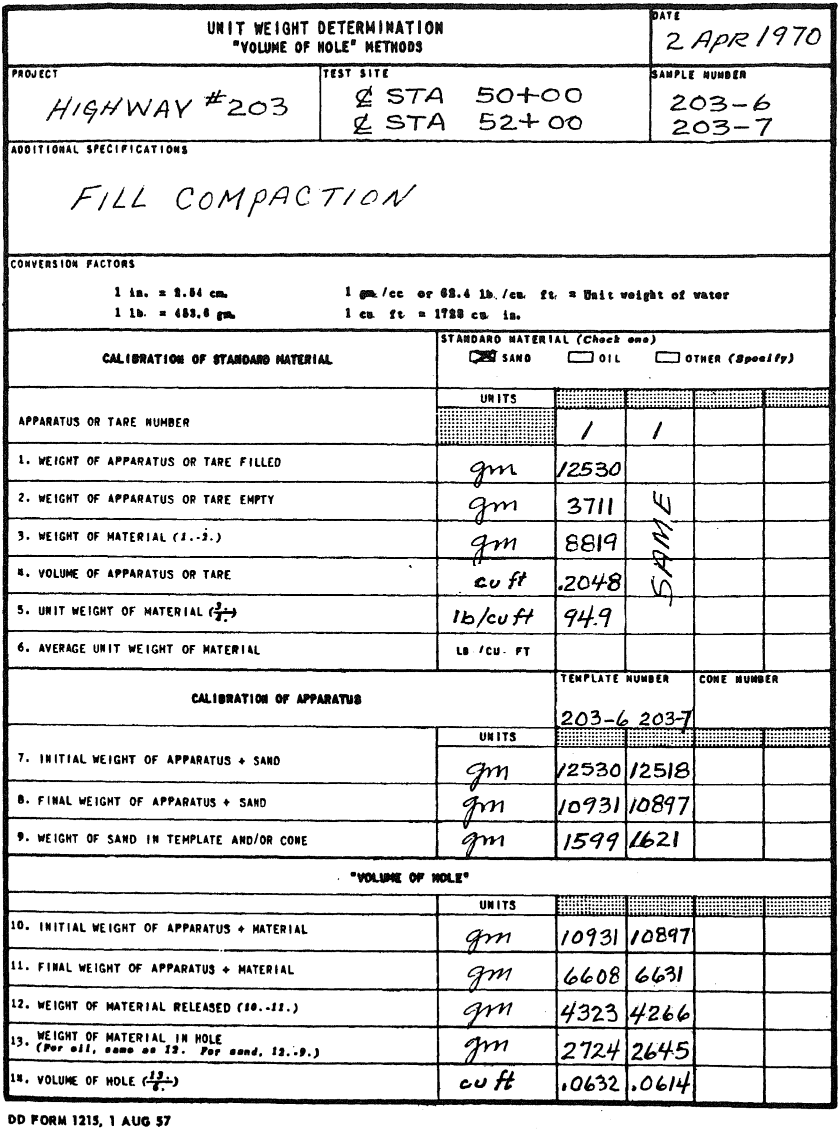
Sand No. 1 **Batch No.** A **Date** ____

	1	2	3	Avg
Weight of apparatus filled with water, grams	11,295	11,299	11,303	
Weight of apparatus empty and dry, grams	4248	4249	4250	
Weight of water, grams	7047	7050	7053	7050

$$\textbf{Weight of water in pounds} = \frac{\textbf{Weight in grams}}{453.6} = \frac{7050}{453.6} = 15.542 \textbf{ pounds}$$

$$\textbf{Volume of apparatus} = \frac{\textbf{Weight water in pounds}}{62.3} = \frac{15.542}{62.3} = 0.249 \textbf{ cubic feet}$$

Figure 2-90. Sample data sheet, calibration of sand-cone density apparatus.



UNIT WEIGHT DETERMINATION
"VOLUME OF HOLE" METHODS

DATE 2 APR 1970

PROJECT HIGHWAY #203

TEST SITE
℄ STA 50+00
℄ STA 52+00

SAMPLE NUMBER
203-6
203-7

ADDITIONAL SPECIFICATIONS
FILL COMPACTION

CONVERSION FACTORS
1 in. = 2.54 cm.
1 lb. = 453.6 gm.
1 gm./cc or 62.4 lb./cu. ft. = Unit weight of water
1 cu. ft = 1728 cu. in.

CALIBRATION OF STANDARD MATERIAL

STANDARD MATERIAL (Check one) ☒ SAND ☐ OIL ☐ OTHER (Specify)

	UNITS				
APPARATUS OR TARE NUMBER		1	1		
1. WEIGHT OF APPARATUS OR TARE FILLED	gm	12530			
2. WEIGHT OF APPARATUS OR TARE EMPTY	gm	3711	SAME		
3. WEIGHT OF MATERIAL (1.-2.)	gm	8819			
4. VOLUME OF APPARATUS OR TARE	cu ft	.2048			
5. UNIT WEIGHT OF MATERIAL ($\frac{3.}{4.}$)	lb/cu ft	94.9			
6. AVERAGE UNIT WEIGHT OF MATERIAL	LB/CU. FT				

CALIBRATION OF APPARATUS

TEMPLATE NUMBER 203-6 203-7
CONE NUMBER

	UNITS				
7. INITIAL WEIGHT OF APPARATUS + SAND	gm	12530	12518		
8. FINAL WEIGHT OF APPARATUS + SAND	gm	10931	10897		
9. WEIGHT OF SAND IN TEMPLATE AND/OR CONE	gm	1599	1621		

"VOLUME OF HOLE"

	UNITS				
10. INITIAL WEIGHT OF APPARATUS + MATERIAL	gm	10931	10897		
11. FINAL WEIGHT OF APPARATUS + MATERIAL	gm	6608	6631		
12. WEIGHT OF MATERIAL RELEASED (10.-11.)	gm	4323	4266		
13. WEIGHT OF MATERIAL IN HOLE (For oil, same as 12. For sand, 12.-9.)	gm	2724	2645		
14. VOLUME OF HOLE ($\frac{13.}{6.}$)	cu ft	.0632	.0614		

DD FORM 1215, 1 AUG 57

Figure 2-91. Data sheet, soil density in place, sand cone method.

2-71. Sand-Displacement Test Procedure

The preparation and calibration up to this point have dealt solely with the equipment. The next procedures are performed on the soil at the site where the test is required.

a. Site Preparation. Prior to the density determination, a hole must be dug and a soil sample taken.

(1) *Setting the template.* The ground surface where the density is to be determined is prepared by clearing and leveling. No attempt should be made to pack or smooth the surface with a spatula, trowel, or other tool. The template tray (*j*, fig. 2-89) should be seated flush on the surface, especially around the center hole. Any spaces on the inside edge under the plate should be sealed with modeling clay. Nails should be forced through the holes in the tray to hold it firmly in position.

(2) *Surface calibration.* As mentioned in paragraph 2-70*c*(3), the surface irregularities inside of the metal tray must be accounted for. To do this, the volume of space between the inner surface of the upper cone and the test surface must be checked.

(*a*) The apparatus with its sand fill (para 2-70*c*(2)) is used for this calibration.

(*b*) With the valve closed, turn the apparatus over, large cone down, and place it over the hole in the metal tray. Be careful not to jar or shake the container and cause packing of the sand.

(*c*) Open the valve carefully and permit the sand to fill the space under the cone.

(*d*) Close the valve and weigh the apparatus and remaining sand. Record this weight (line 8, fig. 2-91). The difference in weight between the starting and final values (lines 7 and 8) is the weight of sand in the tray and cone (line 9).

(*e*) After making the calibration, recover as much of the sand from the tray as feasible without disturbing the tray or the soil in the hole. Brush the remaining sand particles lightly from within the tray.

(3) *Digging the sample.* The next procedure is to dig into the soil through the center hole in the tray. A hole is required which is about 6 inches deep and approximately the same diameter as the hole in the tray. The digging of samples in nonplastic coarse grained soils is difficult, especially in keeping the walls of the hole undisturbed. Driving a chisel or other tool with a hammer should not be tried unless no other method works and then only near the center of the hole. The inside of the hole should be kept as free of pockets and sharp protuberances as possible. Coarse grained material with plastic fines or fine grained soils are easier to dig. Care should be taken to remove all loose particles from the hole and to see that all the removed particles are included in a container whose weight has been previously determined and recorded (line 23, fig. 2-92). Weigh the sample and container immediately and record the weight (line 15). Keep the lid on the container as much as possible to prevent excessive moisture loss until it can be weighed. The container should be marked for later identification when the soil moisture content is determined.

b. Volume-Of-Hole Determination. After the soil sample has been removed from the hole, the sand-density procedure is as follows:

(1) Place the sand-cone apparatus on the metal plate over the hole. Do not jar or shake the container to cause compaction of the sand.

(2) Open the valve and allow the sand to fill the hole in the ground and the space under the cone.

(3) When the sand stops flowing, close the valve, remove the apparatus, and weigh it and the remaining sand. Record this weight (line 11, fig. 2-91).

(4) The weight of sand required to fill the hole equals the weight of the apparatus and sand (line 8 transferred to line 10) minus the final weight (from (3) above) and minus the weight of sand in the template and cone (line 9, from *a*(2)(*d*) above).

(5) The volume of the hole is computed by dividing the weight of sand in the hole by the calibrated density or unit weight (line 6).

c. Density Determination. With the volume of the hole computed the remaining requirements are the moisture content and the dry weights of the soil sample.

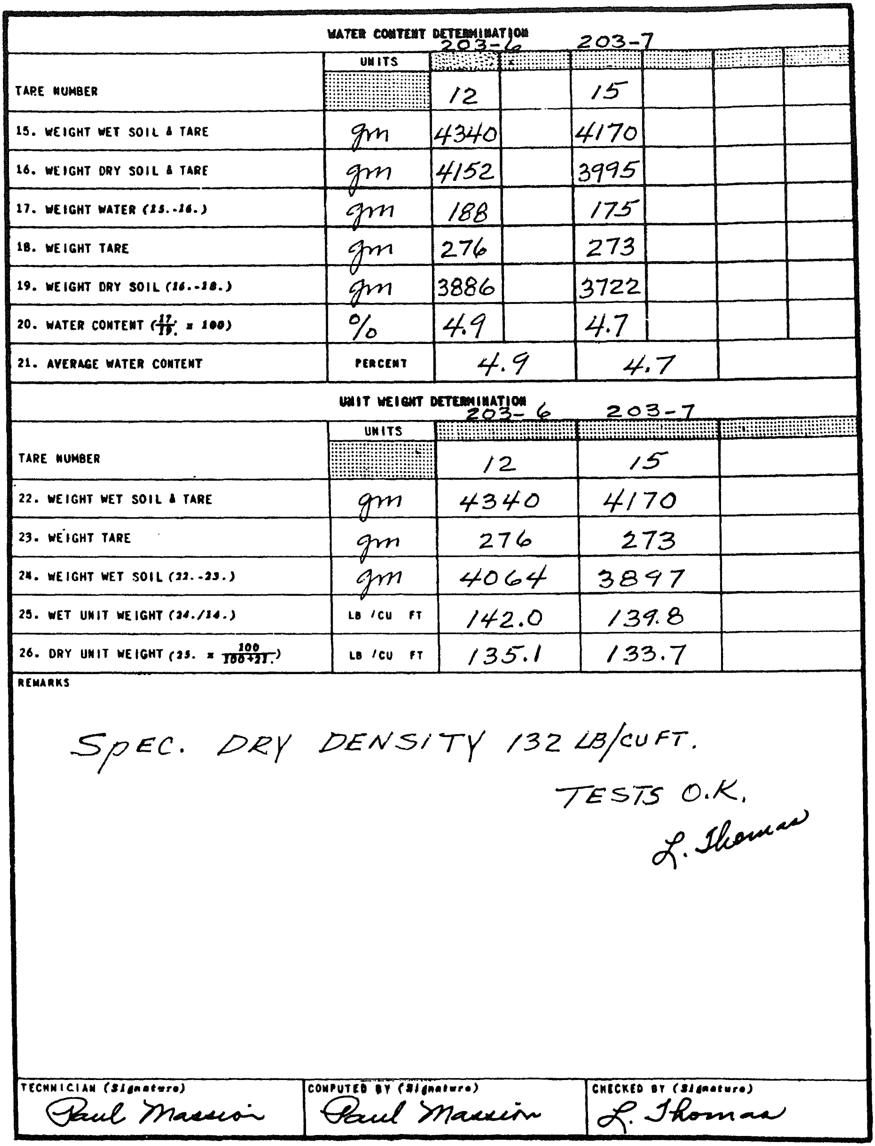
(1) The container with the sample (*a*(3) above) is oven dried, air-cooled, and weighed. Record this weight (line 16, fig. 2-92). The difference (line 15 − line 16=line 17) is the weight of water in the sample. Subtracting the tare weight (line 18) from the dry soil and tare weight (line 16) results in the weight of dry soil (line 19). Moisture content (line 20) is the weight of water (line 17) divided by the dry weight (line 19) and expressed as a percentage.

(2) If drying the entire sample and the container is not possible, an alternate but less desirable method may be used. A specimen selected from the container is weighed and oven dried. Using the weight of the specimen and the entire sample, a percentage is determined and the dry weight of the specimen is translated to give the dry weight of the entire sample.

(3) The wet density or unit weight (line 25) is computed by dividing the wet soil weight (line 24) by 453.6 to convert the grams to pounds, and then by the volume of the hole (line 14). The dry density or unit weight is computed as follows:

$$\text{dry density (line 26)}=\frac{\text{wet density}}{1+\frac{W}{100}}$$

(4) The speedy moisture tester (para 2-55) is recommended when time is critical.



WATER CONTENT DETERMINATION

	UNITS	203-6		203-7			
TARE NUMBER		12		15			
15. WEIGHT WET SOIL & TARE	gm	4340		4170			
16. WEIGHT DRY SOIL & TARE	gm	4152		3995			
17. WEIGHT WATER (15.-16.)	gm	188		175			
18. WEIGHT TARE	gm	276		273			
19. WEIGHT DRY SOIL (16.-18.)	gm	3886		3722			
20. WATER CONTENT ($\frac{17.}{19.}$ x 100)	%	4.9		4.7			
21. AVERAGE WATER CONTENT	PERCENT	4.9		4.7			

UNIT WEIGHT DETERMINATION

	UNITS	203-6	203-7	
TARE NUMBER		12	15	
22. WEIGHT WET SOIL & TARE	gm	4340	4170	
23. WEIGHT TARE	gm	276	273	
24. WEIGHT WET SOIL (22.-23.)	gm	4064	3897	
25. WET UNIT WEIGHT (24./14.)	LB /CU FT	142.0	139.8	
26. DRY UNIT WEIGHT (25. x $\frac{100}{100+21.}$)	LB /CU FT	135.1	133.7	

REMARKS

SPEC. DRY DENSITY 132 LB/CU FT.

TESTS O.K.

L. Thomas

TECHNICIAN (Signature)	COMPUTED BY (Signature)	CHECKED BY (Signature)
Paul Massion	Paul Massion	L. Thomas

Figure 2-92. Reverse of data sheet (DD Form 1215) for sand cone method.

2–72. Liquid-Displacement Methods

a. Oil-Displacement Method. This method is restricted because it cannot be used on pervious soils which will absorb the oil quickly. In addition, residual oil and spillage in a construction area will tend to soften asphaltic concrete placed in the area. However, the method, when used, is similar to sand displacement.

(1) The unit weight of oil must be known in pounds per cubic foot (SAE 40-weight oil is normally used although heavier oil is acceptable. Oil lighter than SAE 30-weight should not be used). A 1000 cc graduate is weighed and its empty weight recorded. The oil is poured into the graduate to the 1000 cc mark, and graduate and oil weighed. The unit weight of the oil (in lb/cu ft) equals the weight of oil, less graduate (in grams), divided by 1000 (cc) and multiplied by 62.36 pounds per cubic foot.

For example: assume 1000 cc of oil weighs 881 g. The unit weight of the oil is—

$$\frac{881}{1000} \times 62.36 = 54.93 \text{ lb/cu ft.}$$

(2) To make the test, the template tray is placed firmly on the prepared ground, and the soil within the hole is dug out and saved. A can is filled with oil and weighed. After the hole is cleaned of loose material, oil from the weighed can is poured quickly into the hole until it is exactly full. A straightedge laid across the hole will define the top edge to which the oil is filled. The can and remaining oil are weighed again. Moisture content of the saved oil taken from the hole is determined. The weight of oil to fill the hole, the unit density of the oil, the wet weight, and the moisture content of the soil are then known. It is now possible to determine the soil density using DD form 1215 (fig. 2–91).

(3) Most of the oil in the hole should be recovered using a suction gun. Care should be taken not to pick up soil with the gun. However, a small amount of soil in the can will not affect future tests if it is allowed to settle to the bottom and only clear oil poured from the top.

b. Water-Displacement Method. The volume of the hole from which a soil density sample is taken can be measured by placing a rubber balloon in the hole and observing the volume of water required to fill the balloon. A water-balloon device is a watertight container with a float attached to a calibrated scale, graduated directly in cubic feet. A balloon is attached to the bottom of the device to make the test. The cylinder in the device is filled with water and the apparatus is placed over the area where the sample is to be removed. The balloon is allowed to fill with water and an initial reading is taken. After the sample is removed from the ground, the device is replaced over the dug hole in the original position. The water is allowed to flow by gravity into the balloon in the hole. By blowing through the hose attached to the device, the air pressure on the water surface is increased and forces the water-filled balloon in the hole to conform to all the contours of the hole. Observing the scale attached to the float results in a reading of the water volume left in the device. Subtract this value from the original reading. The result is the volume of the hole in cubic feet.

2–73. Drive-Cylinder Method

The drive-cylinder method of determining in-place density can be used satisfactorily in moist, cohesive, fine grained soils and in many sands which exhibit tendencies toward cohesiveness. The drive samplers are thin-walled cylinders driven into the soil to obtain an undisturbed sample. These may be driven samplers (para 2–17), or an expedient form of sampler (para 2–20). To obtain an equal sample at all times, some method must be used to cut the sample flush with the ends of the cylinder. This may be accomplished with a steel straightedge sharpened on one edge, or a knife, or with a special cutter used with the soil trafficability sampler. The samples are carefully dug out with a shovel to produce an undisturbed soil sample. A balance, capable of weighing to 0.1 g, is needed to weigh the soil sample in its different states, as well as an oven to dry the sample.

2–74. Procedure Near the Surface

a. The weight of the cylinder (in grams) and the volume (in cubic inches) are determined and recorded prior to starting the test.

b. The soil at the point of testing is prepared by clearing all loose material and exposing a soil surface. The drive-cylinder is placed in position on the surface and driven into the soil so that the top of the cylinder is at least ½ inch below the original surface. The cylinder is dug from the ground by cutting soil away from around the cylinder for several inches and undercutting the cylinder several inches below its lower edge (para 2–17 through 2–20). The sample is trimmed flush with the ends of the cylinder. If the sample is disturbed during trimming or a large particle encountered during driving, the sample is discarded and another one dug.

c. The wet weight of the sample is determined as quickly as possible after the trimming is completed. A moisture content specimen (of about 100 g) is removed from the center of the sample. Using the techniques discussed in paragraphs 2–53 and 2–54, the moisture content is computed.

2–75. Procedure Below the Surface

Samples from below the surface require digging a hole to reach the desired level. Shallow holes may be dug with a shovel until an undisturbed surface is reached. For deeper holes, the auger is used to dig the hole to the required level. The loose material is cleaned out of the hole as thoroughly as practicable. A drive sampler is used that will reach the level, and a sample is taken by driving the top of the cylinder about ½ inch below the surface. The cylinder is removed and the sample trimmed. The determination of wet weight, dry weight, and moisture content follows the usual procedure.

2–76. Calculations

The in-place dry density is expressed as—

$$\frac{\text{dry weight of soil}}{\text{volume of soil}}\ (\text{lb per cu ft})$$

Applying the moisture content to the cylinder specimen, the dry weight is calculated as follows:

$$\text{dry weight}=\frac{(\text{weight of cylinder plus wet specimen}-\text{weight of cylinder})}{1+\left(\frac{\text{moisture content (in \%)}}{100}\right)}$$

The soil volume is computed from measurements of the drive cylinder. The dry weight and soil volume are entered into the formula to obtain the dry density. Note that the drive cylinder is usually constructed to standard specification, and once the volume is computed, it can be used as a standard for the tests. By including necessary conversions, it is possible to multiply the dry weight of the sample in grams by a precomputed factor to obtain the dry density.

For example: a drive cylinder having an inner diameter of 2.58 inches and length of 14.1 inches is used to secure a sample. The empty cylinder weighs 2.65 pounds and the cylinder containing the wet sample weighs 68.93 pounds. Moisture content of the sample is determined to be 6.4 percent. Then the dry density is computed as follows:

$$\text{dry weight}=\frac{68.93-2.65}{1.064}=\frac{66.28}{1.064}=62.2\text{ lb.}$$

$$\text{Volume of cylinder is }\frac{\frac{\pi}{4}(2.58)^2\times 14.1}{144}=0.512\text{ cubic feet}$$

Hence the in-place dry density is

$$\frac{62.2}{0.512}=121.5\text{ pounds per cubic foot.}$$

Section XII. CALIFORNIA BEARING RATIO TEST

2–77. Discussion

The California Bearing Ratio (CBR) method of design for flexible pavements started as an essentially empirical method. Developments and thorough testing have led to the establishment of a distinct pattern for design relations. From this pattern, standard procedures have been evolved for constructing any desired set of CBR curves.

a. The Corps of Engineers adopted the method and, through research and development, made it applicable to the design of flexible pavements for airfields in locations where frost action is not the controlling factor.

b. The test procedure determines the CBR to be used in design of a specific airfield and consists of two principal steps: first, the preparation of soil test specimens, and second, the penetration test performed upon the prepared soil samples. Although a fixed, standardized procedure has been established for the penetration portion of the test, it is not possible to establish one fixed procedure for the preparation of test specimens since soil conditions and construction methods vary widely. The soil test specimens are prepared so as to duplicate the soil conditions existing (or expected to occur later) in the field. For airfields, the method of preparing the test specimens and the number of specimens depend upon the type of airfield, the soils encountered at the site, and other factors. The soil sample should be tested in the laboratory at a density comparable to that obtained during construction. In cases where moisture conditions are favorable and the subgrade will not accumulate moisture approaching a saturated condition, samples should be tested at a moisture content approximating actual moisture conditions expected during the time the road or airfield is used. In all other conditions, samples are laboratory tested in a saturated condition. Although penetration tests are most frequently performed upon laboratory specimens, they also may be performed upon undisturbed soil samples or in the field upon the soil in place.

2–78. The CBR Test

The basic operations for conducting the CBR test are the same regardless of variations in soil conditions and types of construction.

a. The test essentially measures the soil resistance to penetration prior to reaching its ultimate shearing value. It is not exactly a measure of the shearing modulus since the confining effects of the molds do exert some influence. The CBR is designated as a ratio in percent from 0 to 100 with a crushed well-graded (passing through ¾″ sieve) limestone serving as the 100 percent material.

b. Minor variations in the CBR test will cause wide variations in the results. For this reason, the still arise. Material with gravel or stones does not yield entirely satisfactory results. A number of tests must be conducted to establish a reasonable average value.

c. The CBR values range from as low as 3 to as high as 80 depending on the type of soils. The fine grained soils vary from 3 for organic clays to 15 for micaceous or diatomaceous silts and sands. The sand-silt-clay coarse grained combinations range from 10 for the clayey-mixtures to 40 for the gravelly and silty sands. Gravelly soils range from 20 for the clayey group to 80 for the well-graded gravels and gravel sand mixtures. Table B–1 (app B) lists the typical ranges for soils classified under the Unified Soil Classification System.

2–79. Equipment

The equipment used for preparing and testing remolded specimens is listed below. Some of it is illustrated in figure 2–93:

Spacer disk, 2 or 2.5 inches high.

Compaction tamper (para 2–64*a*(3)).

Adjustable stem perforated base plate with tripod support and dial gage (read to 0.001 inch) attachments for measuring soil expansion. One extra base plate is desirable for a set of molds, if the molds are inverted during the test specimen preparation.

Set of slotted annular surcharge weights for loading the cylinder during the test.

Penetration piston with an end area of 3 square inches (1.95 inches diameter) and long enough to pass through the surcharge weights and to penetrate the soil.

CBR screw jack of testing machine capable of forcing the piston into the specimen at a rate of 0.05 inch per minute.

Other general equipment such as mixing bowls, spatulas, straightedges, scales, soaking tank or buckets, oven, and moisture content cans.

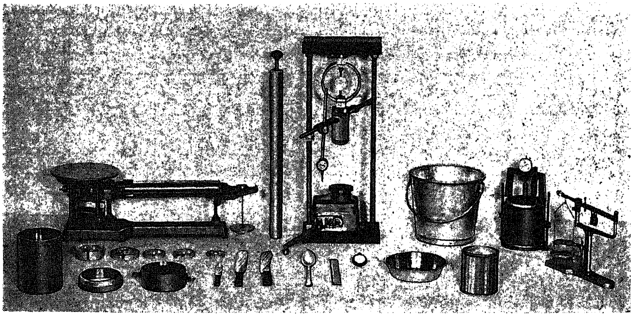


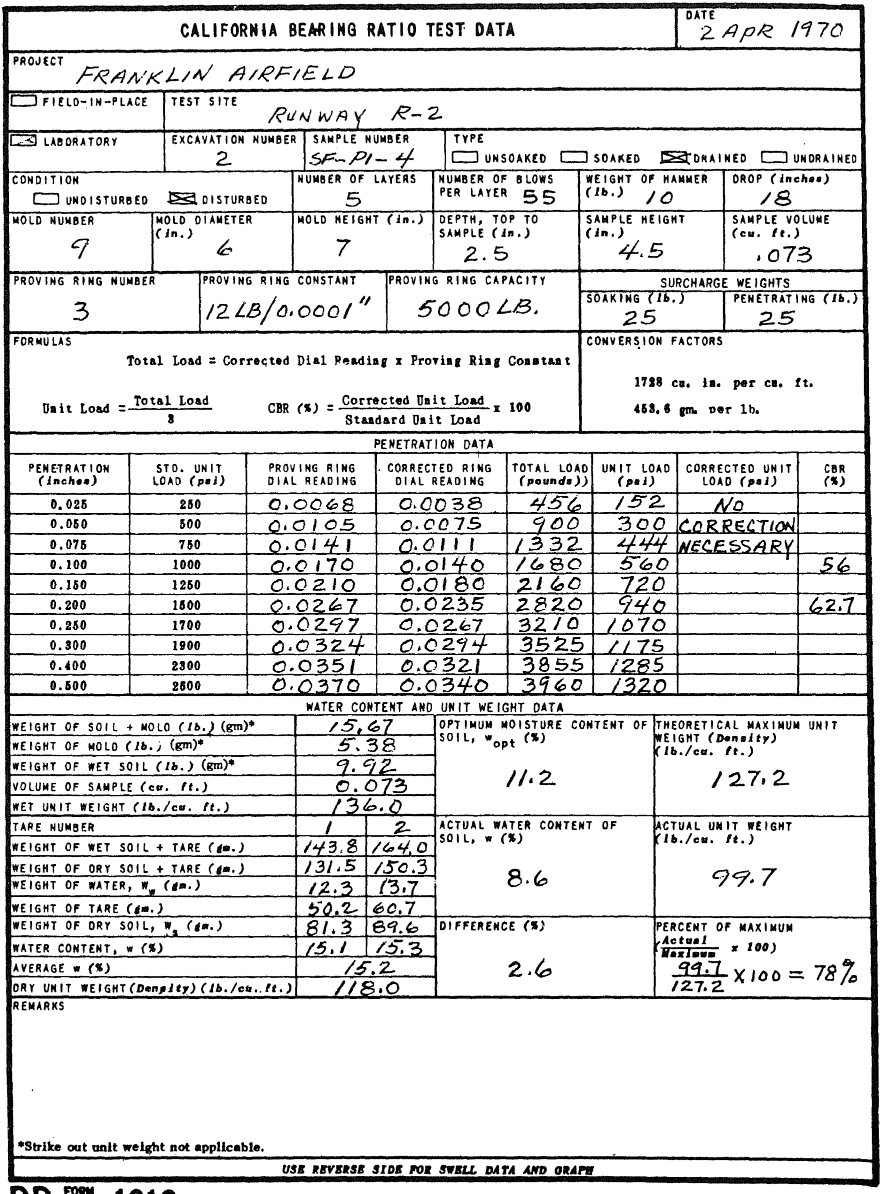
Figure 2–93. Soil test apparatus, laboratory CBR test.

2–80. Preparation of Samples

CBR values are most useful when the specimens tested duplicate the density and moisture content expected in the field. In general, the most critical condition for most materials will exist when the maximum amount of water has been absorbed. For this reason, the CBR test is made on specimens which have soaked for 4 days and while confined in molds under a surcharge equal to the weight of the planned base and pavement.

a. Size of Materials. The soil to be used is air-dried and mixed thoroughly. Artificial drying may be used provided that the sample temperature does not exceed 140° F. The soil should be broken up into individual particles but not pulverized. A representative amount is screened first over the ¾-inch sieve and then the No. 4 sieve. For soils that contain particles over ¾ inch in size, all of the material retained on the ¾-inch sieve is removed and replaced with an equal percentage by weight of material between the No. 4 sieve and the ¾-inch sieve (para (2–64*b*). The sample is recombined and thoroughly mixed.

b. Compaction of Samples. Compaction tests are carried out in accordance with the procedures discussed in paragraph 2–65 and the data entered on DD Form 1212 (California Bearing Ratio Test Data) (figs. 2–94 and 2–95). The compactive effort used and the number of compacted samples required will depend upon the soil type, weight and type of field compaction equipment, and other job conditions. Normally, compactive efforts of 12, 26, and 55 blows per layer (for five layers) are used in each of three successive compaction tests. This permits plotting the moisture, density, and CBR relationship curves.



CALIFORNIA BEARING RATIO TEST DATA

DATE 2 APR 1970

PROJECT FRANKLIN AIRFIELD

☐ FIELD-IN-PLACE

TEST SITE RUNWAY R-2

☒ LABORATORY

EXCAVATION NUMBER 2

SAMPLE NUMBER SF-PI-4

TYPE ☐ UNSOAKED ☐ SOAKED ☒ DRAINED ☐ UNDRAINED

CONDITION ☐ UNDISTURBED ☒ DISTURBED

NUMBER OF LAYERS 5

NUMBER OF BLOWS PER LAYER 55

WEIGHT OF HAMMER (*lb.*) 10

DROP (*inches*) 18

MOLD NUMBER 9

MOLD DIAMETER (*in.*) 6

MOLD HEIGHT (*in.*) 7

DEPTH, TOP TO SAMPLE (*in.*) 2.5

SAMPLE HEIGHT (*in.*) 4.5

SAMPLE VOLUME (*cu. ft.*) .073

PROVING RING NUMBER 3

PROVING RING CONSTANT 12 LB/0.0001"

PROVING RING CAPACITY 5000 LB.

SURCHARGE WEIGHTS

SOAKING (*lb.*) 25

PENETRATING (*lb.*) 25

FORMULAS

Total Load = Corrected Dial Reading x Proving Ring Constant

$$\text{Unit Load} = \frac{\text{Total Load}}{3} \qquad \text{CBR (\%)} = \frac{\text{Corrected Unit Load}}{\text{Standard Unit Load}} \times 100$$

CONVERSION FACTORS

1728 cu. in. per cu. ft.

453.6 gm. per lb.

PENETRATION DATA

PENETRATION (*inches*)	STD. UNIT LOAD (*psi*)	PROVING RING DIAL READING	CORRECTED RING DIAL READING	TOTAL LOAD (*pounds*))	UNIT LOAD (*psi*)	CORRECTED UNIT LOAD (*psi*)	CBR (%)
0.025	250	0.0068	0.0038	456	152	NO	
0.050	500	0.0105	0.0075	900	300	CORRECTION	
0.075	750	0.0141	0.0111	1332	444	NECESSARY	
0.100	1000	0.0170	0.0140	1680	560		56
0.150	1250	0.0210	0.0180	2160	720		
0.200	1500	0.0267	0.0235	2820	940		62.7
0.250	1700	0.0297	0.0267	3210	1070		
0.300	1900	0.0324	0.0294	3525	1175		
0.400	2300	0.0351	0.0321	3855	1285		
0.500	2600	0.0370	0.0340	3960	1320		

WATER CONTENT AND UNIT WEIGHT DATA

WEIGHT OF SOIL + MOLD (*lb.*) (gm)*	15.67	
WEIGHT OF MOLD (*lb.*) (gm)*	5.38	
WEIGHT OF WET SOIL (*lb.*) (gm)*	9.92	
VOLUME OF SAMPLE (*cu. ft.*)	0.073	
WET UNIT WEIGHT (*lb./cu. ft.*)	136.0	
TARE NUMBER	1	2
WEIGHT OF WET SOIL + TARE (*gm.*)	143.8	164.0
WEIGHT OF DRY SOIL + TARE (*gm.*)	131.5	150.3
WEIGHT OF WATER, W_w (*gm.*)	12.3	13.7
WEIGHT OF TARE (*gm.*)	50.2	60.7
WEIGHT OF DRY SOIL, W_s (*gm.*)	81.3	89.6
WATER CONTENT, w (%)	15.1	15.3
AVERAGE w (%)	15.2	
DRY UNIT WEIGHT (*Density*) (*lb./cu. ft.*)	118.0	

OPTIMUM MOISTURE CONTENT OF SOIL, w_{opt} (%) 11.2

THEORETICAL MAXIMUM UNIT WEIGHT (*Density*) (*lb./cu. ft.*) 127.2

ACTUAL WATER CONTENT OF SOIL, w (%) 8.6

ACTUAL UNIT WEIGHT (*lb./cu. ft.*) 99.7

DIFFERENCE (%) 2.6

PERCENT OF MAXIMUM $\left(\frac{\text{Actual}}{\text{Maximum}} \times 100\right)$

$$\frac{99.7}{127.2} \times 100 = 78\%$$

REMARKS

*Strike out unit weight not applicable.

USE REVERSE SIDE FOR SWELL DATA AND GRAPH

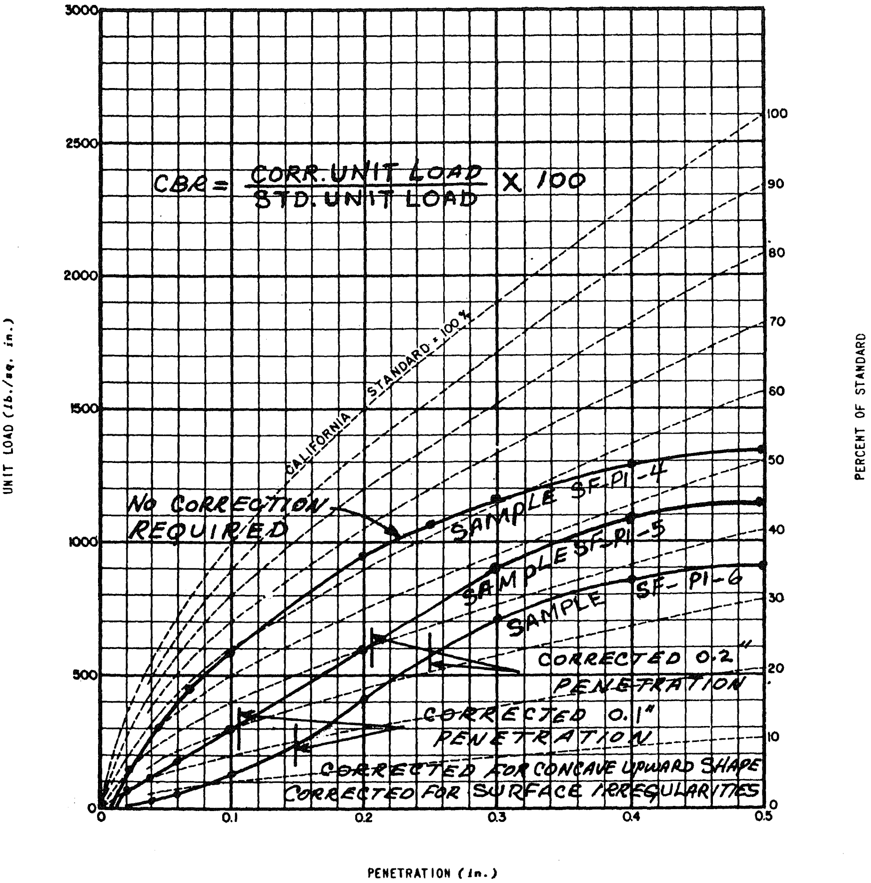
DD FORM 1 AUG 57 **1212**

Figure 2–94. Data sheet, (CBR) test (DD form 1212).

SWELL DATA					
DATE	TIME	DAYS	GAGE READING	SWELL (in.)	SWELL (%)
29 MAR		0	0.304		
30 MAR		1	0.316	0.012	
31 MAR		2	0.328	0.024	
1 APR		3	0.337	0.033	
2 APR		4	0.349	0.045	1.0
		5			

CALIFORNIA BEARING RATIO TEST GRAPH

(Plot test curve below to obtain corrected unit load)



TECHNICIAN (Signature) John Case	COMPUTED BY (Signature) John Case	CHECKED BY (Signature) Charles Layne

B29619

Figure 2–95. Stress-penetration curves (reverse of data sheet (CBR) test, DD form 1212).

Compaction will always be accomplished in the 6 inch diameter CBR mold with a 2- or 2½-inch spacer disk in the mold beneath the sample. Since the mold is 7 inches high, the sample will be 4½ or 5 inches high, depending on the spacer disk used. The compaction tamper is the modified AASHO tamper, which weighs 10 pounds and falls 18 inches.

c. Soaking the Sample. After the sample has been compacted and excess material trimmed away so that the sample is flush with the top of the mold, the perforated base plate and spacer block are removed. A piece of filter paper is placed over the trimmed end (top during compaction) and the perforated base placed on this end. The top of the compacted sample will become the bottom of the soaked sample.

(1) Invert the mold so that the portion next to the spacer block during the compaction becomes the top surface.

(2) Place the adjustable stem with the perforated plate on the top surface with a filter paper underneath.

(3) Apply surcharge weights equal to the expected base and pavement weight (within ±5 lb but *in no case less than 10 lb*). One 5-pound surcharge weight is equivalent to 3 inches of overlying material.

(4) Enter the amount of the surcharge used in the appropriate space on DD Form 1212 (fig. 2–94).

(5) Immerse the mold in water in a bucket or other suitable container so as to allow a free access of water to top and bottom of the specimen, as illustrated in figure 2–96. The adjustable stem extending from the perforated swell plate is set so that the tripod dial can travel in either direction; the sample may consolidate, or it may swell.

(6) Using the tripod and extensometer, obtain an initial reading for swell measurements and record it on the top line on reverse of DD Form 1212.

(7) Normally, the required soaking period is 4 days. A shorter immersion period for pervious soils is permissible, if it is apparent that a maximum degree of saturation has been obtained. Take dial readings at 1-day intervals during the soaking period.

(8) When the soaking period is ended, read the extensometer to determine the amount of consolidation or swell. The amount of swell or consolidation is computed as a percent of initial height of specimen. More than 3 percent swell is considered excessive.

d. Draining the Samples. After soaking, free surface water is removed and the specimen allowed to drain for about 15 minutes prior to conducting the penetration portion of the test. Care must be taken not to disturb the surface of the specimen during the removal of free water; it probably will be necessary to tilt the sample. The surcharge weight and the perforated swell plate are removed, and the specimen eighed. This weight is entered in the lower portion the data sheet. For computing percent saturation, the tare weight of the mold must be determined before the start or at the conclusion of the test; the weight of dry soil is calculated by using moisture content samples from the top and bottom of the mold, after the penetrating portion of the test is completed.

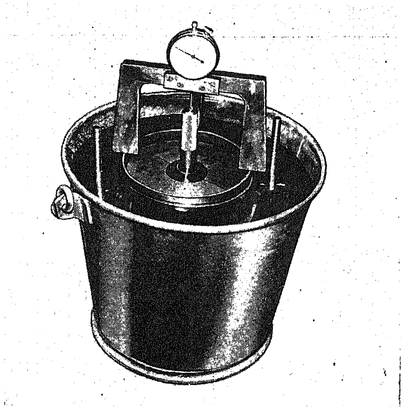


Figure 2–96. Soaking CBR test sample.

2–81. Penetration Test

The penetration test is always the same, regardless of variations in specimen preparation. The following procedure also is applicable to undisturbed mold samples and in-place tests:

a. Assembly of Apparatus. The parts of the CBR apparatus including proving ring, penetration piston, and penetration dial are assembled as shown in figure 2–97. Three proving rings having capacities of 2,000, 5,000, and 7,000 pounds are included in the equipment for this test. The total load on the penetration piston should be estimated on the basis of the expected CBR and the proper proving ring selected so that its capacity will not be exceeded, since overloading can do permanent damage to a proving ring. Standard unit loads on the piston at 0.3-, 0.4-, and 0.5-inch penetration are 1,900, 2,300, and 2,600 pounds per square inch, respectively. Thus, if the anticipated CBR is 80 percent and the test will be carried to 0.5-inch penetration, the approximate total load on the piston will be $2{,}600 \times 3 \times 0.80 = 6{,}240$ pounds at a penetration of 0.5 inch, since the area of the piston is 3 square inches. Similarly, if the expected CBR is only 10 percent and the test will be carried only to a penetration of 0.3 inch, the approximate total load on the piston at that point will be $1{,}900 \times 3 \times 0.10 = 570$ pounds.

b. Surcharge. A penetration surcharge is applied, and must be sufficient to produce an intensity of loading equal to the weight of base and pavement (as

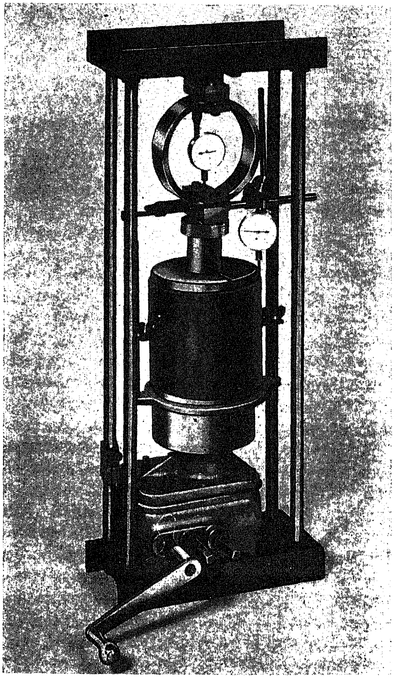


Figure 2-97. Assembled apparatus, CBR penetration test.

described in para 2-80*c*). To prevent the soil being squeezed into the hole of the surcharge weights, one 5 pound annular disk is placed on the sample, the penetration piston is seated, and finally, the remainder of the weights are applied. It should be noted that for laboratory molded samples the penetration test is performed on the bottom of the sample as originally molded, since the sample is inverted before penetration.

c. Seating Piston. The peneration piston is brought into contact with the sample, with sufficient pressure to cause the load dial to register a load of 1 pound. This insures a satisfactory seating of the piston and is considered as the zero load in further tests.

d. Application of Load. Load is applied to the penetration piston by cranking the jack at such a rate that the piston travels at 0.05 inch per minute. The mechanical jack must be calibrated for the rate of 0.05 inch per minute, or the rate of penetration may be determined by comparing the progress of the hand on the penetration dial with the second hand of the clock or watch. Readings of the proving ring dial are made at the following depths of penetration: 0.025, 0.050, 0.075, 0.1, 0.125, 0.150, 0.175, 0.2, 0.250, 0.3, 0.350, 0.4, 0.450, and 0.5 inch. Proving ring dial readings are entered in the appropriate spaces on DD Form 1212. Corrected proving ring dial readings are computed later, and converted to load in pounds using the calibration sheet provided with each proving ring, or the calibration constant engraved on the proving ring block.

e. Moisture Content. The sample is removed from the loading device and a moisture content sample taken from the upper inch beneath the penetration piston and another from the bottom inch of the sample.

f. Penetration. The unit load is obtained in pounds per square inch by the computation indicated in figure 2-94. Using the reverse side of DD Form 1212 (fig. 2-95), the unit load in pounds per square inch (psi) is plotted against depth of penetration in inches. To obtain true penetration loads, it is sometimes necessary to adjust the zero point of the curve to correct for surface irregularities and for the initial concave-upward shape which develops on some soils under certain conditions. The correction is made by extending downward a line tangent to the steepest portion of the curve which extends for a distance of not less than 0.1 inch in the penetration range. The point at which this line intersects the zero load line then becomes the zero penetration point of the curve. The 0.1 and 0.2 inch points are moved to the right on the curve the same distance that the zero point has been advanced to the right. In figure 2-95, necessary corrections have been made to two of the curves shown. Corrected unit loads are entered in the appropriate places on DD Form 1212.

g. California Bearing Ratio (CBR). The correct load values are determined at 0.1 and 0.2 inch penetration. CBR values are obtained by dividing the loads at 0.1 and 0.2 inch by the standard loads of 1,000 and 1,500 psi, respectively. The standard loads of 1,000 and 1,500 psi are always constant and represent the loads required to penetrate a well-graded, minus ¾-inch, crushed limestone sample at a Modified AASHO maximum density. Each ratio is multiplied by 100 to obtain the CBR in percent. The CBR is usually selected at 0.1 inch. If the CBR at 0.2 inch is greater, the test should be rerun. If check tests give similar results (0.2 inch penetration), the CBR at 0.2 inch should be used.

h. Test Results. When the CBR test is completed, the results should contain the following information:

(1) Compaction procedure (Modified AASHO).

(2) Compaction effort (12, 26, or 55 blows/layer).

(3) Molding water content (moisture content prior to soaking).

(4) Density (prior to soaking).

(5) Soaking and penetration surcharges.

(6) Expansion of sample (percent swell).

(7) Moisture content after soaking (saturated condition).

(8) Density after soaking (saturated condition).

(9) Optimum moisture content and maximum dry density by Modified AASHO (CE 55) compaction test (55 blows per layer).

(10) Stress-penetration curve and corrections, if necessary.

(11) Calculations and selection of a corrected CBR value.

2–82. Remolded Sample Test

In determining CBR, remolding of the test soil produces a change in the shearing resistance of some types of soils. Natural clays, for instance, show a decrease in shearing resistance even when the moisture content remains unchanged. This loss in strength is attributed to a breakdown of the absorbed water films and the soil's natural structure. Some soils exhibit a swelling tendency upon remolding and this must be considered. The soils are grouped into three classes with respect to behavior during saturation; cohesionless sands and gravels, cohesive soils, and highly swelling soils. Separate procedures are given for each of the groups.

a. Cohesionless Sands and Gravels. This group includes the (GW), (GP), (SW) and (SP) classifications (para 2–3 and table B1). Cohesionless soils compact readily and specimens should be prepared at high densities and at the range of moisture content anticipated in the field. If soaking does not lower the CBR, it may be omitted from further tests of the same material. In the construction phase, the material should be placed in an almost saturated condition.

b. Cohesive Soils. This group of soils usually falls into the (GM), (GC), (SM), (SC), (ML), (CL) and (OL) classifications (para 2–3 and table B1). They are tested in a manner to show their behavior over the entire range of anticipated moisture content. Compaction curves are developed for 55, 26, and 12 blows per layer. Each specimen is soaked and)enetrated to develop a complete family of curves howing the relationship between density, molding, vater content, and corrected CBR.

c. Swelling Soils. This group of soils usually com)rises the (MH), (CH), and (OH) classifications. The test procedures are the same as for cohesive soils. However, the objectives of the testing program are different. Tests on swelling soils are made to determine a moisture content and unit weight which will minimize expansion. The proper moisture content and density are not necessarily the optimum values as computed from the Modified AASHO compaction tests. Generally, the minimum swell and the highest soaked CBR occur at a moisture content slightly wetter than the optimum value. It may, therefore, be necessary to prepare high-swelling samples for a wider range of moisture and density than normal. The test results can be evaluated by an experienced engineer who will select the proper values required by the design.

2–83. Undisturbed Sample Test

a. Tests on undisturbed samples are used when the base design calls for uncompacted soil, such as highly compressible clay which loses strength upon remolding, or when correlating field in-place tests to the design moisture condition. For this latter condition, duplicate samples should be tested to determine the correction necessary for the in-place tests. The reduction that occurs from 4 days of soaking is applied as a correction to the field in-place test.

b. Care and patience are necessary to maintain the relatively undisturbed samples in this condition (para 2–15). If proper lateral support is not afforded on the sides of the samples, erroneous CBR values will result. In fine grained materials, molds or metal jackets are satisfactory. With samples cut or trimmed from a pedestal, paraffin or a mixture of 10 percent resin and paraffin is used to fill the annular space and offer support. For gravelly soils, the box method is desirable. Wax paper or paraffin is used to cover the sample and prevent moisture loss during transportation to the laboratory.

c. Soaking and penetration tests are performed as outlined (para 2–80 and 2–81) after removal of the paper or paraffin from the end of the specimen and after the surface is leveled (with a thin layer of sand, if necessary).

d. Computations and test results are reported as discussed below (para 2–85).

2–84. Field In-Place Test

Under certain conditions, the field in-place test is used for determining the load-carrying capacity of a soil. Basically, the penetration phase of the test is the same as described in paragraph 2–81.

a. Conditions for Use. The field in-place test is used under any one of the following conditions:

(1) When the in-place density and moisture content are such that the degree of saturation (percentage of voids filed with water) is 80 percent or more.

(2) When the material is coarse grained and cohesionless so that the changes in moisture content will not greatly affect it.

(3) When the material has been in-place and undisturbed for several years. The moisture content in such material does not become constant, but fluctuates within rather narrow ranges. The time required for the moisture content to stabilize cannot be stated definitely, but the minimum time is approximately 3

years. The field in-place test is considered a satisfactory indicator of the load-carrying capacity in this case.

b. Apparatus. Equipment which is needed for the conduct of field in-place CBR tests is as follows:

(1) *Soil test apparatus, field in-place CBR.* This includes a jack to apply the load, calibrated proving rings, penetration piston with extensions, dial gages and support, surcharge weights, and surcharge plate. This apparatus is furnished in a field CBR chest as part of the soils test set (para 2-79). The in-place test is performed in conjunction with a loaded truck to provide resistance for the CBR screw jack.

(2) *Beam kit, field CBR.* This kit includes a loading beam, a penetration beam, and a beam clamp packed in a beam and weight chest as part of the soil test set.

(3) *Other equipment.* This includes the moisture content cans, truck jack, carpenter's level and plumb, stopwatch, and wrenches. These items are all included in the regular soil test set.

c. Preparation. The apparatus is assembled on the rear bumper of a standard Army 2½-ton truck, which is positioned directly over the spot where the test is to be performed.

(1) The truck is jacked up so that no weight rests on the springs. A short section of 6-inch I-beam to bridge the bumpers and a clamp for securing the jack to the beam are provided.

(2) The ground surface is leveled and prepared by removing loose material. The apparatus is assembled and positioned under the bumper (fig. 2-98). The swivel head must be adjusted so that the penetration piston is plumb, and the device locked in position. The dial foot must rest upon a firm, hard object which has a solid foundation far enough from the area affected by the test so as not to be influenced by the reaction of the test through the soil. Except in the case of an extremely high CBR material which is thoroughly interlocked or cemented, this influence will seldom extend beyond a 1-foot radius with sufficient magnitude to be of any consequence.

(3) The steel plate (as an initial 10-pound surcharge weight) should be seated in a layer of fine sand to distribute the surcharge reaction uniformly below all points of the plate. The penetration piston is seated and an initial load of 1 pound is applied to establish a firm bearing. The deflection gages and the load gages are zeroed before starting the test. Surcharge weights are applied to the steel plate to establish the equivalent load intensity of the expected load (table 2-6).

d. Procedure. With the penetration piston set, the surcharge weights added, and the load and penetration dials set to zero, the test can be performed as follows:

(1) Jack the penetration piston at a rate of 0.5 inch per minute.

(2) Read the proving ring (load) readings at the following depths of penetration: 0.025, 0.050, 0.075, 0.100, 0.125, 0.150, 0.175, 0.200, 0.250, 0.300, 0.350, 0.400, 0.450, and 0.500 inch.

(3) Using the proving ring calibration, determine the bearing value (in pounds per square inch).

(4) Plot the penetration versus load deformation curve, with corrections if necessary (fig. 2-95).

(5) Compute the corrected CBR in percent.

(6) Obtain a sample of the soil at the point of penetration for moisture content determination.

(7) Repeat the procedure in (1) through (6) above, two or more times (for a total of three tests) in the same type of soil.

(8) If the results do not show reasonable agreement, three additional tests should be made. The numerical average of the six tests is used as the CBR at that location. Reasonable agreement is designated as the following:

CBR range	*Permissible tolerance*
Less than 10	3
10-30	5
30-60	10
Above 60	unimportant

For example: Test results of 6, 8, and 9 are reasonable and average 8; 23, 18, and 20 are reasonable and average 20. Values below 20 are rounded off to the nearest unit, and above 20 to the nearest 5 units.

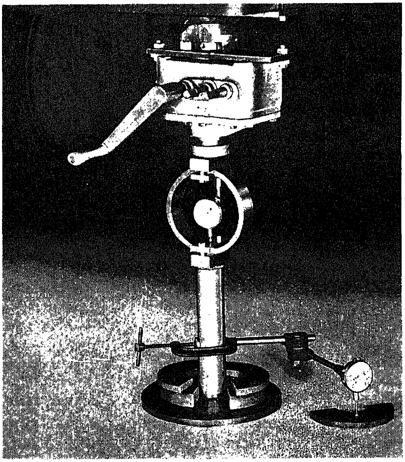


Figure 2-98. Assembled apparatus, field in-place CBR test.

Table 2-6. Surcharge Weights, Depth Versus Density

Depth in.	Wet density, lb/cu ft														
	80	85	90	95	100	105	110	115	120	125	130	135	140	145	150
4	15	15	16	17	18	19	20	21	22	23	24	25	25	26	27
6	22	23	25	26	28	29	30	31	33	34	35	37	38	40	41
8	29	31	33	35	36	38	40	42	44	45	47	49	51	53	55
10	36	39	41	43	45	48	50	52	55	57	59	61	64	66	68
12	44	46	49	52	55	57	60	63	65	68	71	74	76	79	82
14	51	54	57	60	64	67	70	73	76	80	83	86	89	92	**
16	58	62	65	69	73	76	80	84	87	91	95	98	102	105	
18	65	70	74	78	81	86	90	94	98	102	106	110	115	119	
20	73	77	82	86	91	96	100	105	109	114	118	123	127	132	
22	80	85	90	95	100	105	110	115	120	125	130	135	139	145	
24	87	93	98	104	109	115	120	126	131	136	142	147	152	158	
26	95	100	106	112	118	124	130	136	142	148	154	159	165	171	
28	102	108	114	121	127	134	140	146	153	159	165	172	178	184	
30	109	116	123	129	136	143	150	157	164	170	177	184	191	198	

*30-lb surcharge on surface and above line.
**90-lb surcharge is maximum surcharge that will be used for ordinary test. In special cases additional weight may be needed, therefore surcharge weight up to 198 lb has been computed.

2–85. Presentation of CBR Data

The data from the CBR tests and the resulting family of CBR curves represent characteristics for a wide range of field conditions. The CBR data, the molding water content, and the dry density can be combined to present the test results in a highly usable form. This combination is an extension of the compaction curve data and is illustrated in A, B, and C, figure 2–99.

a. Part A, figure 2–99 is the compaction graph (discussed in para 2–65*b*(4)) using the moisture content and density as coordinates. A curve is drawn for each of the three compactive efforts; 55, 26, and 12 blows. The optimum moisture content (OMC) and maximum density are determined as explained. Assume that the lean clay can be compacted to a density of 95 percent of Modified AASHO; then this density is computed from the maximum dry density (A, fig. 2–99) as 116.5×.95=110.6 and the result is plotted on the graph. A horizontal line (dashed on the figure) is drawn through the plotted point until it intersects the drawn curve. In this case, values of ±1½ percent are plotted on either side of the OMC (14.1±1½=12.6 or 15.6 percent). For ease in plotting, the lines have been drawn at 13 and 16 percent until they intersect the 55 blow curve and the 95 percent density line. The shaded area in A, figure 2–99, represents values greater than 95 percent Modified AASHO density and within about ±1½ percent of the OMC. These values will be used later as explained in *c* below.

b. B, figure 2–99, is a graph using the relation of corrected CBR values for the same molding moisture contents as used in graph A. For ease in plotting, the CBR *vs* water content graph is drawn directly above the density graph and uses the same abscissa as the graph A.

c. The desired presentation is a family of curves graph showing the relation of molded density to corrected CBR (C, fig. 2–99) at a constant moisture content. The graph is plotted by selecting several whole percent values of moisture contents below the OMC–

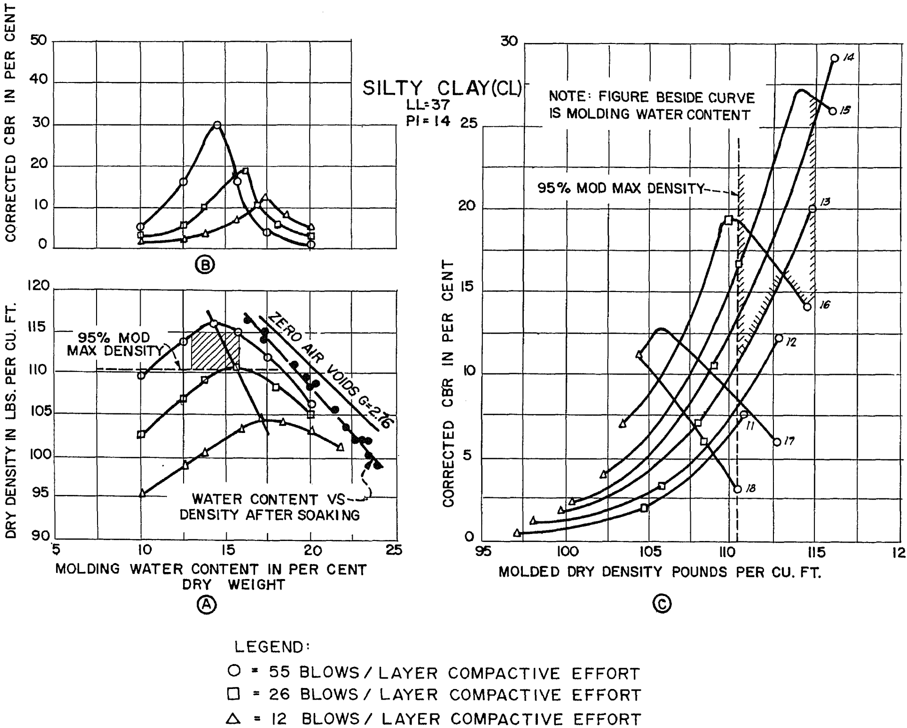


Figure 2–99. Presentation of CBR data.

ercent computed in *a* above (11 percent in the ple). The density (from graph A) and the CBR 1 graph B) corresponding to this selected value letermined for each of the three compactive s.

1) To determine the values, place a straight-or draw a line vertically through the point along ottom edge corresponding to the selected mois-content.

2) Move up the line until it intersects the 12 curve on the density graph (A, fig. 2–99), and the density corresponding to this value.

3) Move up the line to the 12 blow curve on 1 B, and read the CBR corresponding to this

(4) On graph C, plot the CBR *vs* density (read) and (3) above) and mark this point as the 12 point.

(5) Using the same selected moisture line, deter-the CBR *vs* density for the 26 and the 55 blow ts, and plot these values on graph C.

(6) Draw a smooth curve through the three ed points, and mark it with the moisture content ted. As an example, see line marked "11", C, e 2–99.

(7) Repeat this process ((1) to (6) above) for tional moisture contents desired using incre-s of 1 percent until all three points on the curve a definite falling off. Note the 18 percent curve figure 2–99.

(8) Transfer the desired density, in this case the ercent line (110.6 lb per cu ft), and the maximum vable density line (115 lb per cu ft) from graph The hatched vertical lines in graph C indicate values.

d. To interpret the test results from graph C, consider the hatched portion. This illustrates that the CBR can vary from 11 (13 percent moisture; 95 percent density) to 26 (15 percent moisture; maximum allowable density). For design purposes, a CBR of 11 or 12 would be used and the moisture content specified to stay between 13 and 16 percent. Thus, when 95 percent density (or better) is attained in the field with a moisture content maintained between 13 and 16 percent, the CBR would definitely be above its design value.

e. Note how the graph illustrates the effect of moisture content variations on the CBR. A drop to 12 percent would require a maximum compactive effort to reach a CBR of 12. Maintaining this same effort and achieving this same density at 13 percent moisture increases the CBR to 16, and at 14 percent moisture to above 20. At the upper limit (as little as 1 percent increases in moisture (16 to 17 percent) will cause a drop in CBR from 18 to 8 at 95 percent density. At 18 percent moisture, the CBR would decrease to 3. This presentation illustrates the importance of maintaining the moisture content within the specified range until the 95 percent density is reached.

f. Note that while the above example uses a 3 percent moisture content range, a 4 percent moisture content range is considered acceptable. A 4 percent moisture content range would necessitate checking the design CBR for the following moisture content ranges: OMC±2 percent; OMC+1 to OMC−3 percent; OMC−1 to OMC+3 percent; OMC to OMC+4 percent; and OMC to OMC−4 percent. The moisture content range that produces the highest design CBR is the recommended range.

Section XIII. PLATE BEARING TEST

6. Discussion

esigning concrete pavements for roads or air-s, certain features must be considered. These in-e the anticipated loads, the type and amount of ned traffic, the supporting value of the subgrade, the characteristics of the concrete. The antici-d traffic and loading are design factors and are nd the scope of this manual. The materials test-technician may be called upon, however, to per-1 the plate bearing test to determine the modulus ıbgrade reaction, *k*, of the subgrade for a con-e pavement. The modulus of subgrade reaction is load per unit area required to produce a soil de-nation of the subgrade. It is expressed in pounds square inch per inch of deformation.

7. Preparation and Test Procedures

test is performed on representative areas and ections or adjustments can be made by the designer in light of experience or knowledge of the local conditions which can alter the load supporting value of the subgrade. Most soils exhibit a marked reduction in modulus with an increase in moisture content. However, since the design is not too sensitive to small changes in *k*, a bracket or range of values is sufficient.

a. Equipment. The following equipment is needed to conduct the plate bearing test:

Hydraulic jack, 25,000 pound capacity.
Proving ring or hyrdaulic gage.
Bearing plates, 18-inch, 24-inch, and 30-inch diameter by 1-inch-thick or 1½-inch-thick aluminum alloy plate.
Dial indicators, 0.001 inch, three required (with extensometers).
Dial support beam and clamps (A frame for 3-sided support).
Load reaction beam (steel I or H beam).

grade should be stripped free of overburden to the elevation of the subgrade surface which will be used. It should be uncovered for a sufficient area to eliminate any surcharge of confining effects. If the subgrade is to be fill material, a test embankment of about 30 inches in height should be constructed after the necessary stripping. The subgrade should be compacted to the specified densities at optimum moisture content. The test should be made with the soil as close to optimum moisture content as practical. If possible, it is advisable to conduct comparative tests on the subgrade representative of conditions after the pavement has been in place for some time. Methods of correction are available when it is impractical to simulate actual conditions. On granular subgrades, the natural condition of the soil should not be disturbed while the test area is being prepared. Prior to placement of the 30 inch diameter plate, all loose material and boulders which protrude more than 1 inch above the lowest point should be removed and replaced by a layer of sand or plaster of paris mixture to prevent the plate from resting on these points. The thickness of this layer should be held to the absolute minimum needed to prevent point bearing.

c. Loading System. Loads are applied by a hydraulic jack working against a jacking frame and through a steel bearing plate 30 inches in diameter and 1 inch thick (fig. 2–100). The load must be applied in the exact center of the plate to reduce the possibility of tilting the plate during the test. A nest of plates (30 inch, 24 inch, and 18 inch diameter) is used to assist in the centering. The plates must be leveled prior to applying the load. If cribbing is needed, it is best to place such filling members between the top plate and the jack. If a proving ring is used to measure load, it should be placed on top of the jack and a ball joint used between it and the load reaction device. The load reaction device must be long enough so that its supports will be at least 8 feet from the bearing plate. Aluminum-alloy plates which have the same rigidity as the 1-inch steel plates may be used if weight reduction is necessary.

d. Load Reaction Devices. The load reaction from the jack can be taken by bearing against a truck trailer, anchored frame, or any other device of sufficient magnitude for the test. Granular subgrades require a reaction of at least 25,000 pounds while for most cohesive subgrades, 10,000 pounds is sufficient.

e. Deformation Measuring. The movement of the plate due to the applied loads should be measured in three places, spaced 120° apart around the circumference of the 30 inch plate and within ¼ inch of the rim. Dial gages, accurate to 0.0001 inch, will be required for granular subgrades. However, gages accurate to 0.001 inch may be used if time readings are made.

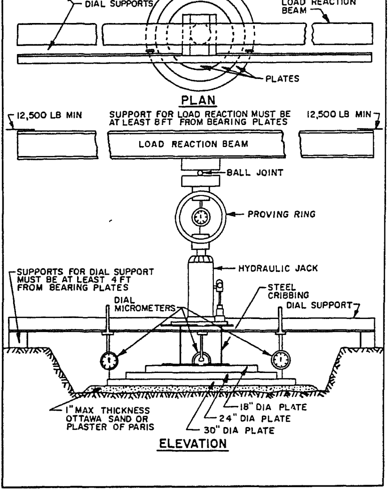


Figure 2–100. Plate bearing test loading.

f. Loading Procedure. The loading system and bearing plate should be seated by applying a load of 707 pounds (1 pound per square inch (psi) on the 30 inch diameter plate) for light load pavements (less than 15 inches thick) or 1,414 pounds (2 psi) for heavy load pavements (15 inch or more). The seating load must be allowed to remain until complete deformation of the subgrade has taken place. The dial gages are read and the values recorded as a zero correction.

(1) An additional 7,070 pound (10 psi) load is applied and held until complete deformation has taken place. Before releasing the load, a value of k'_u (preliminary subgrade modulus) is computed from the average deflation on the rim:

$$k'_u = \frac{10 \text{ psi}}{\text{average deflection in inches}}$$

where the deflection readings on the dials at 10 psi are averaged and the units are psi/inch.

(2) For cohesive subgrades, where k'_u is less than 200 psi/inch the test is considered complete and the load may be released. Some clay soils may require a time-settlement curve to determine when complete deformation has been reached. The load is held until the rate of deflection is less than 0.0002 inch per minute (0.002 inch in 10 minutes) which indicates that most of the consolidation has taken place. A field plot of average dial reading versus time, on arithmet-

ical coordinates may be made to show settlement rates.

(3) For granular subgrades or some cohesive subgrades with a k'_u of 200 psi per inch or more, the load is applied in successive increments of 3,535 pounds (5 psi) to a maximum load of 21,210 pounds (30 psi). The load is not released between increments. Each increment loading is held for at least 16 minutes and until less than 0.0002 inch per minute deformation is read. The final load (30 psi) is held until practically complete deformation is reached, but not less than 16 minutes. In general, this will be when the rate of deformation is less than 0.0001 inch per minute (0.001 in/10 min) and most of the consolidated has occurred.

g. Unit Load-Deformation Curve. A curve of unit load *vs* deformation should be plotted for materials with a k'_u of 200 psi per inch or more. The plotted points are joined with a smooth curve (fig. 2–101). If the curve approximates a straight line between 10 psi and 30 psi loadings, and goes through the "0" point, no correction is needed. If it does not approximate a straight line or the straight line does not extend through the "0" point, a straight line is drawn through the origin and parallel to the straight line or the straight line portion of the curve (fig. 2–102a). If no portion approximates a straight line, the correction line is drawn parallel to the average slope as determined by at least three points in the area of least curvature (fig. 2–102b). A new value of deformation is then read at the 10 psi point on the corrected curve.

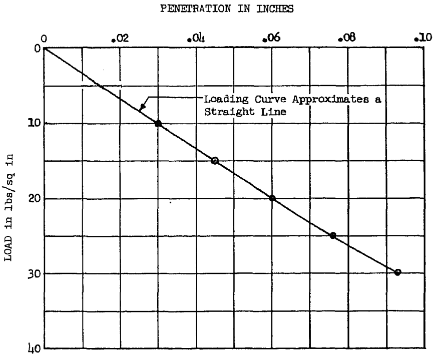
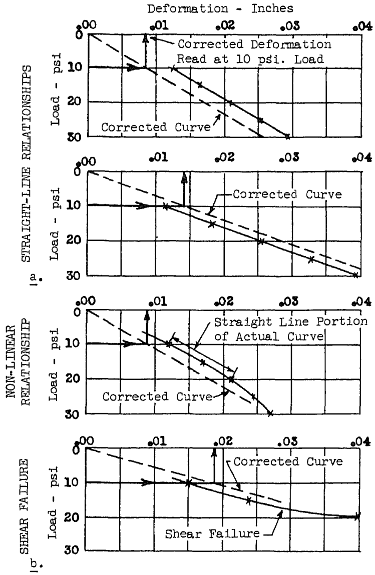


Figure 2–101. Load-deformation curve from plate bearing test.



Note: Corrected Curve May Lie Above or Below Actual Curve

Figure 2–102. Zero corrections to load-deformation curves.

2–88. Evaluation of Results

a. Deflection Correction. There is a certain amount of bending in the bearing plate when load is applied, even when a nest of plates is used, and bending results in a greater deflection at the center than on the rim. Since the deflection is measured at the rim, the measurement is smaller than if taken at the center and the k'_u is higher. For k'_u of less than 100 psi per inch, the bending correction is negligible and can be omitted. For k'_u between 100 and 200, the correction graph (fig. 2–103) is used. A k'_u greater than 200 requires a load-deformation curve to be drawn and a new value of k'_u determined. This value is used to enter the curve and to read the corrected value (k_u), the subgrade modulus corrected for plate deflection.

b. Saturation Correction. For a better evaluation, a correction for subgrade saturation should be considered. The most applicable method is an adaptation of the consolidation test. The samples are selected from the location where the tests are to be performed. The correction is determined by loading two samples

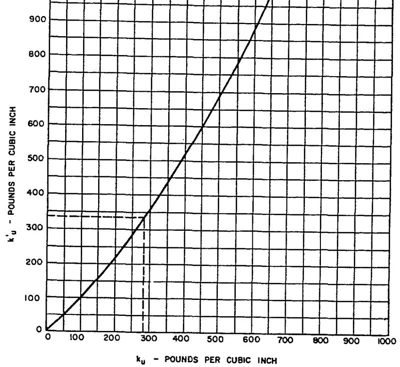


Figure 2–103. Plate bending correction for k_u.

of the undisturbed subgrade material, one with the moisture content in-place (sample A) and the other (sample B) with the undisturbed subgrade in a saturated condition. The same seating load (para 2–87*f*) used in the plate bearing test is applied to both specimens. When the vertical dial becomes stable, a "zero" reading is taken and an additional 10 psi load is applied (total of 11 or 12 psi). After the dial becomes stable again, the reading reflects the deformation under a 10 psi load. Certain soils may swell under saturation and cause the material to squeeze up over the restraining ring when pressure is applied. The top of both of these specimens, A and B, should be trimmed about 1/16 inch to allow for swelling, so that the height of both specimens is the same at the start of the test. The correction for saturation is applied in proportion to the deformation of both specimens under a load of 10 psi. The value of the corrected subgrade modulus (k) is—

$$k=\frac{\quad}{d_s}\times k_u$$

Where:

k=subgrade modulus corrected for saturation and bending of the plate,

k_u=subgrade modulus, uncorrected for saturation, but corrected for bending of the plate when necessary,

d_s=deformation of a standard specimen under a unit load of 10 psi in inches, and

d=deformation of a specimen at field conditions under a unit load of 10 psi in inches.

Note. The value of $\frac{d}{d_s}$ is limited to less than one.

c. Interpretation of Corrections. k_u is rarely used in design without a correction. For coarse granular soils, k_u may be used without a correction if compaction requirements are met. Cohesive soils in arid regions with a low water table may have actually smaller saturation corrections than determined by the test. If examination of other pavements in the area indicates no loss in support at the joints due to infiltration or leakage and the moisture content is lower than optimum for compaction, the uncorrected (for saturation) value of k_u may be used. In some instances, the correction does not fulfill the intended purpose and is not satisfactory. Two examples of these soils are: cohesionless or nearly cohesionless silts and fine sands which "pump"; and uniform, poorly-graded and poorly-compacted sand that tends to shift at pavement joints. For these conditions, a k value is assumed for the design.

d. Correlation of Subgrade Modulus and CBR. If time and equipment are not available for a full scale loading test, an approximate k value may be determined from the CBR value. This must be done for each soil type encountered. Table B–2 (app B) shows typical k and CBR design values based on the type of classified subgrade soil.

Section XIV. UNCONFINED COMPRESSION TEST

2–89. Discussion

a. The unconfined compression test is used widely to estimate the shearing strength of clay soils. It cannot be performed on cohesionless soils which cannot stand unsupported. The test is most applicable to saturated clay soils, and as described in this paragraph, is performed upon relatively undisturbed samples at their natural moisture contents. In many soils the test is also run as remolded on samples of the same soils at the same moisture content; this procedure is not described.

b. Description. The unconfined compression test is a simple test usually performed on a small cylindrical sample (1–1½ inches in diameter and 2–3 inches high). It resembles the compression test on a concrete cylinder except that the sample and loads are much smaller. The principal information obtained is the value of the compressive unit stress which corresponds to failure of the cylinder, or to 15 percent strain (reduction in height). This value, designated as q_u, is expressed in pounds, or tons, per square foot. The unconfined compression strength is assumed to be equal to twice the shearing strength (cohesion) of such soils. In the form of an equation, this relationship is stated as $q_u=2c$, in which c is the cohesion and q_u is the unconfined compressive strength. The relationship between stress and strain (unit deformation, or in this case, unit shortening), may also be determined during this test. Since the test is made on a sample which must stand by itself with no support and is made immediately after the sample is placed n the test machine, the shearing strength is regarded as a minimum. The use of the test value is, therefore, conservative in many instances.

2–90. Apparatus

a. Basically, the testing is done by loading the sample in the frame provided for the CBR test. Samples used in the test normally are secured with the soil trafficability sampler. Apparatus for conducting the test is assembled from components found under soil test apparatus (para 2–79), penetrometer, plus such other items as may be necessary. The assembled apparatus for the unconfined compression test is shown in figure 2–104. Moisture contents are determined by oven-drying of total specimen.

b. The CBR frame is placed, with jack down, on a table in position to turn the crank and preferably clamped in place. The sample will be placed directly on the jack plunger or on a metal plate which will in turn rest firmly on the plunger plate.

c. A cone penetrometer proving ring is selected from the soil trafficability apparatus. The penetrometer dial (0.001 inch sensitivity graduated from 0 to 300) is then removed and the 0.0001 inch sensitivity dial is substituted. It must be emphasized that, when the unconfined compression test is completed, care must be taken to return the proper dial to the proving ring, since the penetrometer's dial indicator is calibrated for one particular ring only. If the face of the penetrometer is graduated so that 0 to 300 equals exactly 360° of the dial face, it may be assumed that a deflection of 0.1 inch equals 150 pounds; therefore, a deflection of 0.001 inch equals 0.15 pound. However, the readings taken frequently will be in the lower range of the ring's capacity and much greater accuracy will be assured by a recalibration of the ring with the 0.0001 inch dial in place. The ring may be calibrated simply for the range needed for the unconfined compression test (seldom more than 100 pounds total load except for very stiff or hard clays). This is accomplished by attaching the ring vertically in the vise using a convenient thread on the penetrometer shaft and balancing known weights on it or an adapter above it. Calibrated weights may be used or weights prepared by weighting on the solution balance. A series of weights should be used as, for example, ½, 1, 2, 4, 6, 8, 10, 20, 30, and so forth pounds and the corresponding dial readings recorded. Some penetrometer rings have been made whose dials have been calibrated to the ring. Therefore 360° on the dial will not be equal to precisely 300 cone index, but slightly more or less. If such a ring is used with the 0.0001-inch dial, the ring must be calibrated for the dial.

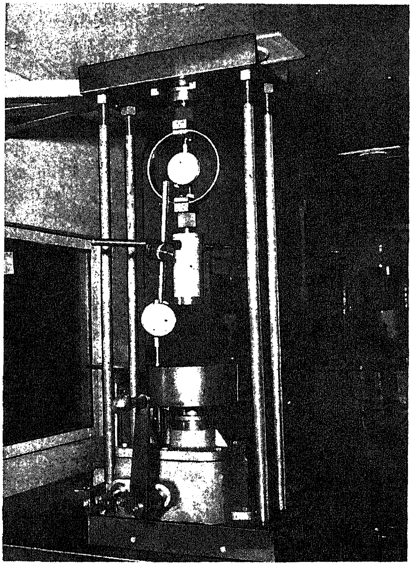


Figure 2–104. Assembled apparatus, unconfined compression test.

d. The recalibrated ring is attached to the CBR ring-to-frame adapter and to the field CBR piston by means of the two penetrometer ring adapters. Piston extensions may be used as needed. The ring dial should be set at zero with the piston to be used hanging free.

e. The strain or deformation of the sample is read from a 0.001-inch-sensitivity dial attached to the dial clamp on the CBR frame in position to read the travel of the jack upward. A zero reading is recorded as the sample, being lifted upward on the jack, comes into contact with the CBR piston above. It should be ascertained that no play whatsoever is present between the ring-to-frame adapter and the frame. If play is found to exist, the adapter should be forced tightly into the clamp by using paper shims.

2–91. Test Procedures and Calculations

As previously indicated, the sample used for an unconfined compression test normally is obtained with the soil trafficability sampler. See paragraph 2–18 for sampling procedures.

(1) Any protective coating or wrapping used to preserve the sample must be carefully removed before testing. The sample obtained by this sampler is 1.87 inches in diameter and 6.0 inches long. Under expedient conditions, samples may be cut from a chunk sample. A minimum size of cylindrical sample in such cases is 1 inch in diameter and 2 inches high. In any case, the height-diameter ratio should be about 2. The sample should be handled very carefully and quickly throughout the test.

(2) The sample should be tested immediately after being extruded from the sample tube, or cut from a chunk sample. The ends of the sample must be examined and trimmed, if necessary, to make certain that they are plane and parallel to each other and perpendicular to the longitudinal axis of the sample. The height of the sample and its cross-section area must be accurately known; the area may be determined by measuring the circumference with a flexible tape and then computing the diameter. If tape is not available, a strip of paper about ½ inch wide is wrapped around the sample, marked for length, and measured. The average of at least three measurements should be used for both the height and area. DD Form 1213 (Unconfined Compression Test) is provided for the unconfined compression test (fig. 2–105).

(3) If desired, the weight of the sample may be determined before testing. This is done to permit computation of the unit weight of the material.

(4) The sample is placed in the loading frame, the loading apparatus adjusted, and both the vertical dial and proving ring dial set at zero. The sample is loaded by turning the jack handle at the rate of 0.05 inch per minute. This is the same rate as required for the CBR penetration test. During the early stages of the test, readings on both dials should be taken corresponding to every 0.01-inch movement of the vertical dial. Later, as the stress-strain curve (fig. 2–106) begins to flatten, readings may be taken at 0.02-inch and, still later, at 0.05-inch intervals in the vertical dial readings. Readings of both dials are recorded in the appropriate space on the data sheet. Stop the test when the axial load remains constant or when 20 percent axial strain has been produced. Record the duration of test, in minutes, to peak strength (time to failure), type of failure (shear or bulge), strain plot sheet, figure 2–106.

(5) After the sample has failed, place the entire specimen or a representative portion thereof in a preweighed container and determine the moisture content in the usual way.

b. Calculations. Necessary calculations are indicated on the data sheet (fig. 2–105).

(1) The unit strain, E, is computed by the equation, $E=\frac{\Delta L}{L_0}$ where ΔL is the change in length of the specimen, in inches, determined from the vertical dial readings and L_0 is the initial length of the specimen, in inches.

(2) The average corrected area, A in square feet, which corresponds to any rate of strain is computed from the expression, $A=\frac{A_0}{1-E}$ in which A_0 is the initial area of the test sample, in square feet.

(3) The total axial load, P, in pounds is determined by multiplying the quantity, proving ring reading minus initial proving ring reading, by the calibration factor for the proving ring or by the use of a calibration curve.

(4) The unit stress is determined by dividing the axial load, P, in pounds, by the corresponding corrected area, A, in square feet.

(5) The stress-strain curve is drawn as indicated in figure 2–106 by plotting the strain or deformation, $\frac{\Delta L}{L_0}$ in percent, on the horizontal scale and the unit stress, P/A, in pounds per square foot, on the vertical scale.

(6) The unconfined compressive strength, q_u is normally taken to be the peak value of P/A on the stress-strain curve. As previously noted, the shearing strength, $c=\frac{1}{2}q_u$. In the example shown (fig. 2–106), c=795 pounds per square foot.

c. Typical Values. Typical values of the unconfined compressive strength of clays of various consistencies are given in table 2–7.

Table 2–7. Typical Values of Unconfined Compressive Strength of Clay Soils

Consistency	*Unconfined compressive strength pounds per square foot*
Very soft	Less than 500
Soft	500–1,000
Medium	1,000–2,000
Stiff	2,000–4,000
Very stiff	4,000–8,000
Hard	More than 8,000

UNCONFINED COMPRESSION TEST

DATE 2 APR 1970

PROJECT BUILDING K-3	EXCAVATION NUMBER 1	SAMPLE NUMBER K-B1-1
PROVING RING DIAL NUMBER 3	PROVING RING NUMBER 3	CALIBRATION CURVE NUMBER 6
PROVING RING CONSTANT, K_r 0.15LB/0.0001"	VERTICAL DIAL NUMBER 2	RATE OF LOAD APPLICATION 0.05"/MIN

FORMULAS

Area $= \frac{\pi D^2}{4} = \frac{C^2 D}{4}$ Corrected Area, $A = \frac{A_0\ (sq.\ ft.)}{1 - E}$ Unit Strain, $E = \frac{\Delta L}{L_0}$ Unit Stress $= \frac{P\ (lb.)}{A\ (sq.\ ft.)}$

INITIAL MEASUREMENTS ON SAMPLE				AVERAGE AREA, A_0	
	DIAMETER, D (*in.*)	CIRCUMFERENCE, C (*in.*)	AREA (*sq. in.*)	SQ. IN.	SQ. FT.
TOP	1.860	5.84	2.742	2.746	0.0191
CENTER	1.868	5.87	2.746	HEIGHT, L_0 (*in.*)	
BOTTOM	1.871	5.89	2.750	3.45	

ELAPSED TIME (*min.*)	PROVING RING DIAL READING (*0.0001 in.*)	LOAD, P (*lb.*) (*Ring Dial x K_r, or from calibration curve*)	VERTICAL DIAL READING (*in.*)	VERTICAL DIAL DIFFERENCE, ΔL (*in.*)	UNIT STRAIN, E (*in./in.*)	CORRECTED AREA, A (*sq. ft.*)	UNIT STRESS (*lb./sq. ft.*)
0	0.0	0.00	0.900	0.00	0.00	0.0191	0
	25.3	3.80	0.890	0.01	0.26	0.0192	199
	59.9	8.98	0.880	0.02	0.52	0.0192	463
	74.9	11.24	0.870	0.03	0.78	0.0193	582
	149.3	22.39	0.860	0.04	1.04	0.0193	1160
	189.6	27.25	0.850	0.05	1.31	0.0194	1410
	200.6	30.11	0.840	0.06	1.57	0.0194	1552
	206.1	30.91	0.830	0.07	1.83	0.0195	1585
	206.7	31.00	0.820	0.08	2.09	0.0195	1590
	206.4	30.97	0.810	0.09	2.35	0.0196	1580
	201.2	30.18	0.800	0.10	2.61	0.0196	1540
	197.7	29.65	0.790	0.11	2.87	0.0197	1505
	193.9	29.08	0.780	0.12	3.13	0.0197	1476
	191.4	28.71	0.770	0.13	3.39	0.0198	1450
	188.1	28.21	0.760	0.14	3.67	0.0198	1425
3.0	184.4	27.66	0.750	0.15	3.92	0.0199	1390

WATER CONTENT (%) 39.2%	UNCONFINED COMPRESSIVE STRENGTH, q_u (*lb./sq. ft.*) 1590

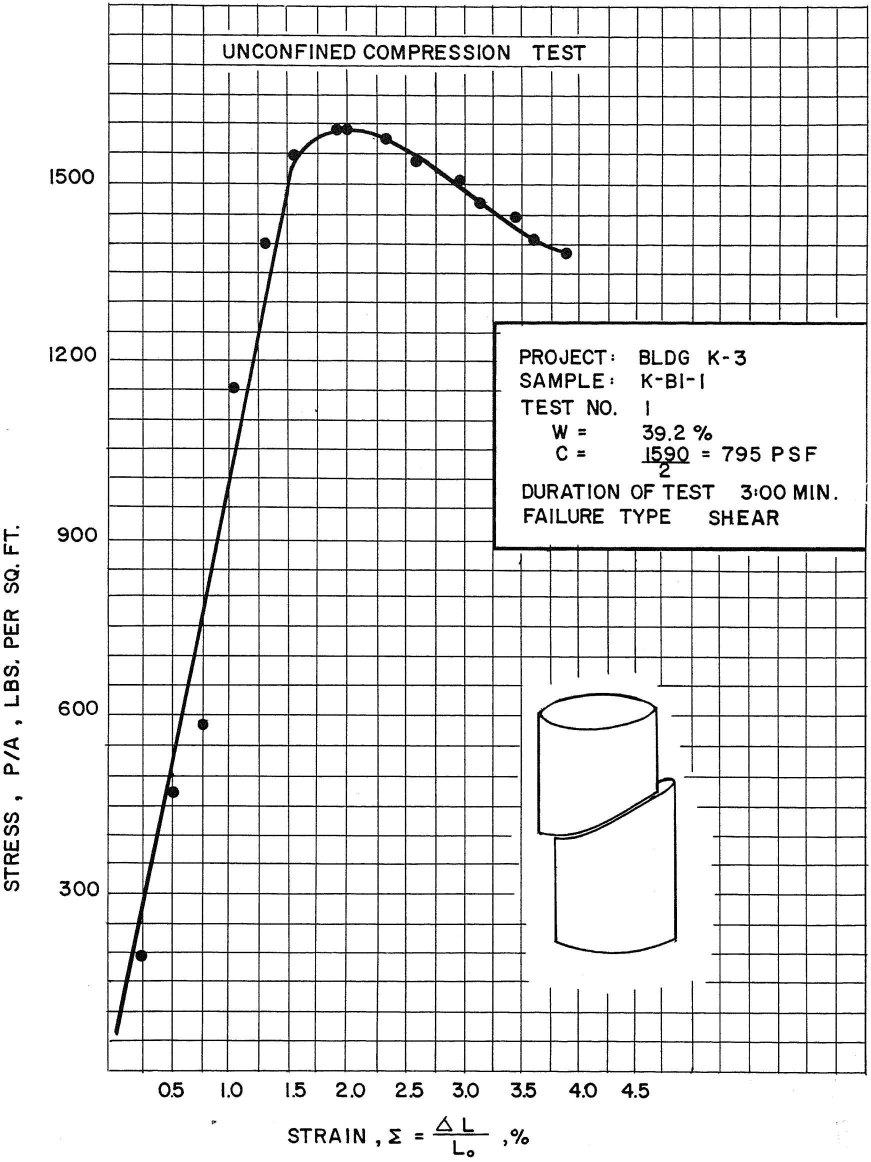
REMARKS AND CALCULATIONS (*Use reverse side, if more space is needed*)

TECHNICIAN (*Signature*) Paul Manin	COMPUTED BY (*Signature*) Paul Manin	CHECKED BY (*Signature*) Arthur Mason

DD FORM 1 AUG 57 1213

GPO 931154

Figure 2-105. Data sheet, unconfined compression tests.



PROJECT: BLDG K-3
SAMPLE: K-BI-I
TEST NO. I
W = 39.2 %
C = $\frac{1590}{2}$ = 795 PSF
DURATION OF TEST 3:00 MIN.
FAILURE TYPE SHEAR

Figure 2–106. Stress-strain curve, unconfined compression test.

Section XV. SOIL TRAFFICABILITY TESTS

2-92. Description

Soil trafficability is the capacity of a soil to support cross-country movement of military vehicles.

a. One commonly used method of determining the trafficability of a soil is by actual contact tests made on the soil in-place with a cone penetrometer and remolding test equipment. These tests are described below. The estimating of soil trafficability from a knowledge of topography, soil types, and weather conditions, by methods for predicting vehicle mobility on both flat and sloping terrain, and by methods of indicating trafficability data on maps or aerial photographs are beyond the scope of this manual.

b. Certain terms which are related to soil trafficability are defined as follows:

(1) *Bearing capacity.* The ability of a soil to support a vehicle without undue settlement of the vehicle.

(2) *Traction capacity.* Ability of a soil to resist the vehicle-tread thrust required for steering and propulsion.

(3) *Cone index.* An index of the shearing resistance of soil obtained with the cone penetrometer. It represents resistance to penetration into the soil of a 30° right circular cone with ½-square-inch base area (actually, load in pounds on cone area in square inches).

(4) *Remolding.* The change or working of a soil by traffic, or by a remolding test. Remolding may have a beneficial, neutral, or detrimental effect on soil strength.

(5) *Remolding index.* The ratio of remolded soil strength to original strength.

(6) *Rating cone index.* The measured cone index multiplied by the remolding index of the same soil layer.

(7) *Critical layer.* The soil layer in which the rating cone index is considered a significant measure of trafficability. Its depth varies with weight and type of vehicles and the soil profile, but it is normally the layer lying 6 inches to 12 inches below the surface.

(8) *Vehicle cone index.* The index assigned to a given vehicle that indicates the minimum soil strength in terms of rating cone index required for 50 passes of the vehicle. The vehicle cone indexes, which are applicable to all types of military vehicles, are not given in this manual.

(9) *Stickiness.* The ability of a soil to adhere to vehicles.

(10) *Slipperiness.* Low traction capacity of a soil's surface owing to its lubrication by water or mud.

c. Both bearing and traction capacities of soils are functions of shearing resistance.

(1) Shearing resistance, as measured by the cone penetrometer, is expressed in terms of cone indexes. Since the strength of a soil may increase or decrease when worked, remolding tests measure the gain or loss of soil strength to be expected under traffic. A comparison of the rating cone index with the vehicle cone index indicates whether the vehicle can negotiate the given soil condition.
For example: If a soil has a cone index of 120 and a remolding index of 0.60 in its critical layer, its strength may be expected to fall to 120 times 0.60, or 72, under traffic. Accordingly, it is considered that such a soil is not trafficable for vehicles with vehicle cone indexes greater than 72.

(2) Both stickiness and slipperiness may hamper the mobility of vehicles under certain conditions. These items are not expressed quantitatively, and are normally troublesome only when associated with soils of low bearing capacity. It may be further noted that, except in a few unusual circumstances, the trafficability of a fine-grained soil will vary with the weather. Rainy periods will cause an increase in moisture, an increase in stickiness and slipperiness, and a decrease in strength; dry periods will have the opposite effects. For this reason, trafficability characteristics measured on a given date cannot be applied later, unless full allowance is made for the changes in soil strength caused by weather.

2-93. Equipment and Adjustment

a. Description. The cone penetrometer is the principal instrument in evaluating soil trafficability. A complete soil trafficability test set, in backpack for portability, is shown in figure 2-107. It consists of a 30 degree right circular cone (base area ½ square inch) mounted on one end of a graduated staff or rod with a measuring device attached to the other end. When the cone is forced slowly into the soil, the measuring device records the amount of force required to move the cone. This penetrating force is considered to be an index of the shearing resistance and is called the cone index of the soil. The cone index refers to some given plane of reference such as the soil surface or some given distance below the soil surface. Readings are first taken when the top of the cone is just even with the soil surface, and then at successive 1, 2, or 6 inch increments as the cone penetrates further into the soil. The 6 inch increments are indicated on the shaft and any other increment can be premarked.

b. Apparatus. The issue cone penetrometer (fig. 2-108) uses two 18 inch extension rods (to provide either an 18 or 36 inch length), a proving ring, a dial gage, and a handle. In use, the dial is mounted inside the proving ring. The amount of force applied to

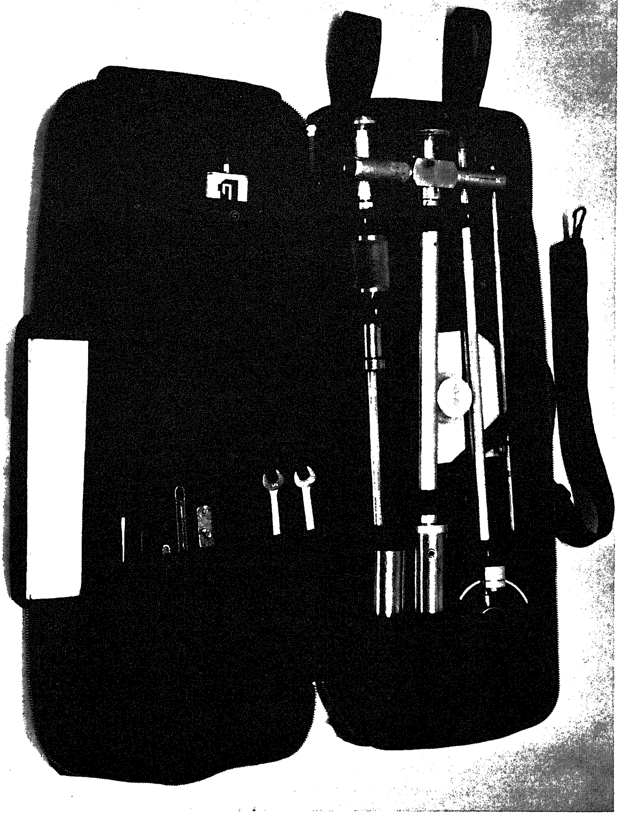


Figure 2–107. Test set, soil trafficability.

move the cone into the soil is indicated on the dial which has a range from 0 to 300 pounds per square inch (0–150 lb). Maintenance and adjustment are as follows:

(1) The penetrometer needs little care beyond keeping it clean and well oiled, keeping the joints tight, and checking the calibration.

(2) Particular care should be taken to see that no grit is caught between the extensometer arm of the dial gage and the lower mounting block.

(3) If either or both mounting blocks loosen and become movable, they should be positioned to lie on a diameter of the ring and then retightened. If this happens, the proving ring calibration should be checked before using the instrument.

(4) The point of the cone may become rounded through use. This may affect the accuracy of the instrument, hence an excessively worn or deformed cone should be replaced.

(5) The proving ring should not be subjected to overstressing, deformation, hard knocks, or extreme changes in temperature. If treated properly, the proving ring will hold its calibration for the life of the instrument.

(6) The micrometer dial gage is a sensitive instrument which must be protected against rough usage. It should never be immersed in water, and if used in rainy weather, should be wiped dry as soon as possible. When transporting the instrument, the dial should be cushioned by wrapping it in a cloth or paper.

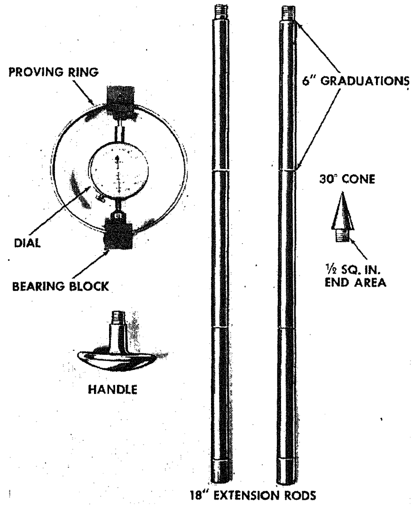


Figure 2–108. Cone penetrometer.

c. Airfield Penetrometer. In addition to its use for soil trafficability, the cone penetrometer is a useful instrument in theater of operations airfield evaluation and design. It must be emphasized that the cone penetrometer is at no time a substitute for the laboratory or in-place CBR methods. However, it may be used when hasty conditions prevail and time is a factor. The cone penetrometer may be used to obtain an airfield index which is a consideration in airfield design. A modified version of the penetrometer permits evaluation of higher indexes than the standard model. Also, a penetrometer specially devised for the Air Force and called the airfield penetrometer can be used.

(1) *Modified penetrometer.* The modification to the described cone penetrometer consists of an adapter attachment which replaces the lower removable half of the staff. It consists of a ⅜-inch staff attachment with a 0.2 square-inch end-area cone mounted on the lower end of the staff. When the 0.5 square-inch base is used, the airfield index is obtained by dividing the cone index by 50, or

$$\text{Airfield index} = \frac{\textit{cone index}}{50}$$

With the 0.2 square-inch cone, the airfield index is obtained by dividing the cone index by 20, or

$$\text{Airfield index} = \frac{\textit{cone index}}{20}$$

(2) *Airfield penetrometer.* This instrument uses a built-in spring-loaded measuring device in place of the proving ring and dial gage. The penetrometer consists of a 30° right circular cone with an end area of 0.2 square inch mounted on a graduated staff. The spring housing and handle are on the other end of the staff. The instrument may be disassembled into three main pieces, each less than 13 inches long, for transportation. The handle forms part of the largest piece and protects the cone. The other two pieces are extension staffs. The entire unit weighs approximately 3 pounds.

(*a*) The load indicator consists of two separate sets of interwound coils which act as a single spring. In use, the spring is stretched (tension) in direct proportion to the applied load. The measuring device located on the spring housing is an indicator which moves up as the force is applied.

(*b*) The technique for measuring with the airfield penetrometer is very simple. Force is applied to the handle until a slow steady downward movement occurs. The indicator on the barrel is read at the moment the base of the cone enters the ground (surface reading), and at any other desired depth when the corresponding depth mark reaches the surface. The reading is made by shifting the line of vision from the depth mark to the load indicator just a moment before the desired depth is reached.

(*c*) When the required depth of penetration is at some distance below the surface, either one or both extension staffs are inserted between the spring housing and the cone. The staffs are marked at 2 inch intervals.

d. Adjustment. The proving ring supplied with the cone penetrometer is calibrated by the manufacturer. This calibration should be accurate throughout the life of the instrument, unless the ring receives a severe jolt, the dial gage is abused, an excessive load is applied to the ring, or the ring is overstressed in some other way. If recalibration is necessary, the procedure given below may be used:

(1) Remove the handle and rod.

(2) Place the lower mounting block of the ring assembly on a smooth, horizontal surface.

(3) Check the mounting block alinement. Both blocks should be on a diameter ring. A drafting triangle or a carpenter's square may be used in this operation.

(4) Seat the extensometer arm of the dial firmly on the lower block with sufficient travel of the arm available for the full range (approx. 0.1 inch deflection) of the proving ring. The dial can be moved up or down by adjusting the two nuts on the threaded stud which holds the gage in position. Both nuts should be tight when in final position.

(5) Zero the dial by rotating its face so the "O" is under the needle.

pounds, marking or noting the position of the needle on the dial after the addition of each load increment. Any of the following loading methods may be used.

(*a*) Dead weights may be added to the top of the ring assembly. If a plate is used to hold the weights, its weight should be considered in the first 10 pound load.

(*b*) Any of the load machines commonly used in laboratory work may be used to apply the load.

d. Take a dial reading just as the base of the cone

(*c*) The ring assembly may be placed on a set of platform scales and the load increments applied by a jack and measured with platform scales.

(7) Remove load in 10 pound increments, noting the position of the needle after the removal of each increment.

(8) The load run should be made at least twice, using the average of the needle position for each increment as the final point.

(9) Some variation in needle position will occur, but will not be significant.

(10) When 10 pound increments have been established on the dial, they may be marked 20, 40, up to 300, and then subdivided into four subintervals. Each interval should be subdivided separately, since the scale arcs of the various 10 pound intervals are not necessarily the same.

2–94. Test Procedure

The step-by-step procedure in using the cone penetrometer is as follows:

a. The penetrometer must be inspected before use to make sure that all nuts, bolts, and joints are tight and that the dial gage stem contacts the proving ring bearing block.

b. Allow the penetrometer to hang vertically from its handle. It should read zero, and when a 150 pound load is applied, it should read 15.

c. Place the hands over each other on the handle in order to minimize eccentric loading of the proving ring and to help keep the rod vertical (fig. 2–109). Force is applied by pressing the chest against the hands until slow, steady, downward movemnt occurs.

d. Take a dial reading just as the base of the cone is flush with the ground surface. Continue the slow, steady, downward movement and take successive dial readings at appropriate intervals, usually at 6 inch intervals, as each 6 inch graduation on the rod enters the ground. A normal rate of progression will yield the usual five readings involved in penetrating to 24 inches in about 15 seconds. An assistant should be provided to record the readings taken by the operator.

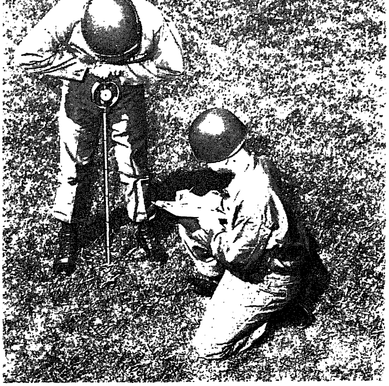


Figure 2–109. Method of using cone penetrometer.

e. The following precautions should be observed in the use of the cone penetrometer:

(1) The instrument must be kept vertical.

(2) Readings higher than the 150-pound capacity of the dial should not be attempted, since this might overstress the proving ring.

(3) If the dial capacity is exceeded at less than 24 inches of penetration, the operator should assure himself that the cone is not striking an isolated rock fragment by making another penetration nearby.

(4) The instrument should never be withdrawn by the ring but always by the rod or the handle.

f. A data sheet should be prepared in advance showing location of observations, date, weather conditions, names of operator and observer and the value of the readings at the 6-inch depth intervals.

2–95. Remolding Test

The remolding index of a soil is an expression of the ability of a soil to retain its original strength under the traffic of a vehicle.

a. Apparatus. Equipment which is required for determining the remolding index includes the following:

(1) Trafficability remolding equipment consisting of a cylinder, approximately 2 inches in diameter and 9 inches long, mounted on a base, and a 2½-pound drophammer.

(2) Penetrometer, with a 36 inch shaft, used to measure soil strength in the cylinder before and after remolding.

(3) Soil trafficability sampler used to obtain the soil sample and to place it in the remolding cylinder.

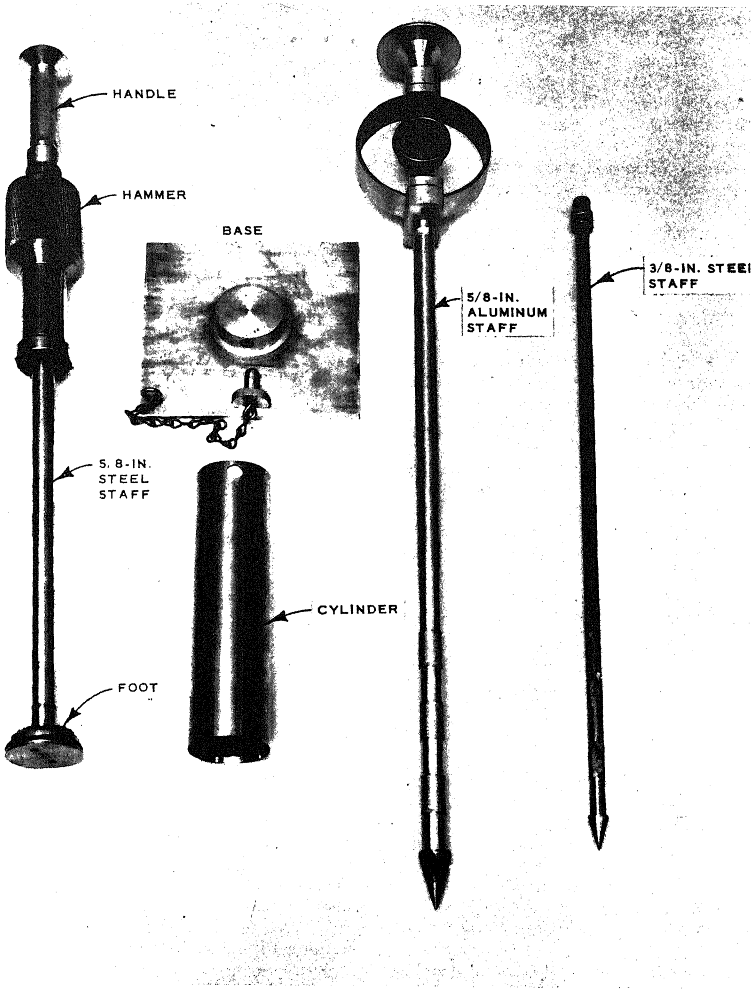


Figure 2–110. Remolding test equipment.

(4) The equipment discussed in (1) and (2) above is illustrated in figure 2–110.

b. Test Procedure. The test procedure for determining the remolding index is as follows:

(1) Take a sample with the soil trafficability sampler (para 2–18).

(2) Eject the sample directly into the remolding cylinder (fig. 2–111).

(3) Push the sample to the bottom of the cylinder with the foot of the drophammer.

(4) Using the penetrometer, take cone index readings as the base of the cone enters the soil surface and at each inch to a depth of 4 inches (fig. 2–112).

(5) Apply 100 blows with the drophammer falling 12 inches (fig. 2–113).

(6) Repeat the cone index readings ((4) above).

penetrated to the full depth. In this case, record a reading of 300 pounds per square inch for each inch below the last reading.

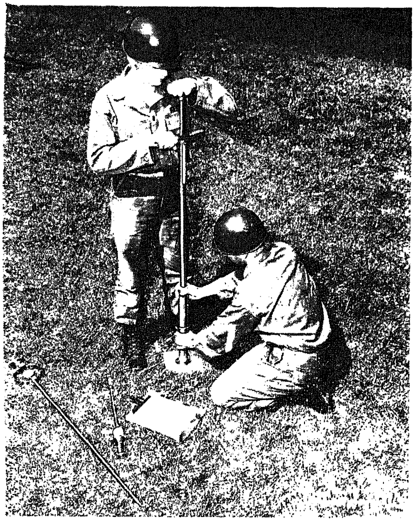


Figure 2–111. Transferring sample into remolding cylinder.

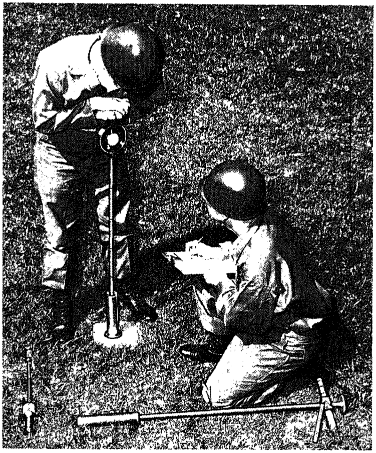


Figure 2–112. Taking cone index readings.

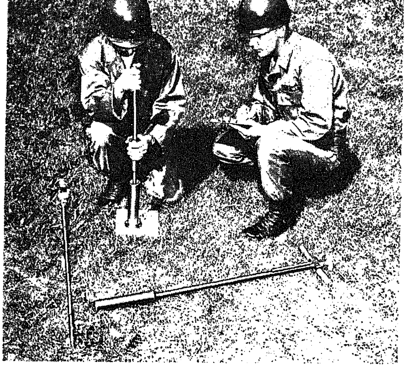


Figure 2–113. Remolding the sample.

c. Calculations. The sum of the five cone index readings after remolding, divided by the sum of the five cone index readings before remolding, gives the remolding index. The remolding index is a ratio of the remolded soil strength to the original soil strength and may be either greater or less than 1.00. Confinement in the small cylinder prevents a direct comparison of cone index in the cylinder with cone index in the ground. However, the ratio determined by the test can be applied along with the cone index of the soil to determine the in-place cone index. The rating cone index is the product of the cone index and the remolding index for the same soil layer. *For example*, if the cone index of a soil in-place is 85 and the remolding index is 0.80, the rating cone index is $85 \times 0.80 = 68$. Thus, the cone index of this soil can be expected to change from 85 to 68 under the action of traffic.

2–96. Typical Results

Probable ranges of the cone index, the remolding index, and the rating cone index are given in table 2–8 for four general groups of soils. The general effects of stickiness and slipperiness are noted in the table, together with some general comments relative to trafficability characteristics of the soils contained in each group. More extensive discussion of the uses of soil trafficability test results is given in TM 5–330, chapter 9.

Table 2-8. Trafficability Characteristics of Soils in Wet Season

Group	Soils	Unified Soil Classification System	Probable cone index range	Probable remolding index range	Probable rating cone index range	Slipperiness effects	Stickiness effects
A	Coarse-grained, cohesionless sands and gravels	GW, GP, SW, SP	35 to 100	Not applicable	Not Applicable	Slight to none	None
B	Inorganic clays of high plasticity, fat clays	CH	55 to 165	0.75 to 1.35	65 to 140	Severe to slight	Severe to slight
C	Clayey gravels, gravel-sand-clay mixtures	GC	85 to 175	0.45 to 0.75	45 to 125	Severe to slight	Moderate to slight
	Clayey sands, sand-clay mixtures	SC					
	Gravelly clays, sandy clays, inorganic clays of low to medium plasticity, lean clays, silty clays	CL					
D	Silty gravels, gravel-sand-silt mixtures	GM	85 to 180	0.25 to 0.85	25 to 120	Moderate to slight	Slight
	Silty sands, sand-silt mixtures	SM					
	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	ML and CL-ML					
	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH					
	Organic silts and organic silty clays of low plasticity	OL					
	Organic clays of medium to high plasticity, organic silts	OH					

Table 2-8. Trafficability Characteristics of Soils in Wet Season — Continued

Comments
Will support continuous traffic of tracked military vehicles or all-wheel-drive trucks with high flotation tires. Performance will increase with a decrease in tire pressure. Moist sands are good, dry sands only fair. Wheeled vehicles with standard tires may be immobilized in dry sands.
Usually will support more than 50 passes of military vehicles. Going will be difficult at times.
Often will not support 40 to 50 passes of military vehicles, but usually will support limited traffic. Going will be difficult in most cases.
Usually will not support 40 to 50 passes of military vehicles. Often will not permit even a single pass. Going will be difficult in most cases.

Table 2-8. continued

CHAPTER 3
BITUMINOUS MIXTURES

Section I. INTRODUCTION

3–1. Description

Bituminous pavements are a mixture of well graded *mineral aggregates*, *mineral filler*, and a bituminous cement or *binder*. They are used as the top portion of a flexible payment structure to provide a resilient, waterproof, load-distributing medium which protects the base course from detrimental effects of water and the abrasive action of traffic. This chapter discusses the materials used in the construction of bituminous surfaces, and the methods of testing these materials and the mixes prepared from them. The design considerations, such as bearing capacity and thickness of pavement, are described in TM 5–330. Mixing and placing operations, as well as the considerations for preparing the mixes, are fully described in TM 5–337.

3–2. Aggregate

The aggregate may consist of crushed stone, crushed or uncrushed gravel, slag, sand and mineral filler or a combination of some of these materials. Aggregates normally constitute 90 percent or more by weight of bituminous mixtures, and their properties have a very important effect upon the finished product.

3–3. Bituminous Cements

a. The bituminous cement is the adhesive agent in the bituminous mixture and may be either a tar or an asphaltic material. The asphalts are generally obtained from the distillation of crude petroleum (fig. 3–1). Asphalt may also be obtained from natural deposits in Trinidad and elsewhere. The tars are obtained from the destructive distillation of bituminous coal (fig. 3–2). The functions of a bituminous cement are to hold the aggregate particles together and to seal the surface which then resists the penetration of water. All bituminous materials possess these qualities.

b. The bituminous cements are available in several forms suitable for different procedures of mixing or application under wide variations in temperature. Some asphalts and tars are solid or semisolid at room temperatures. Other grades are relatively viscous (thick) liquid at room temperatures and all become liquid at higher temperatures. Mixing bitumens with petroleum solvents or water produces *cutbacks* or *emulsions* that are liquid at atmospheric temperatures. Such liquid asphalts and tars are used for cold mixes or applied as sprays in building pavements.

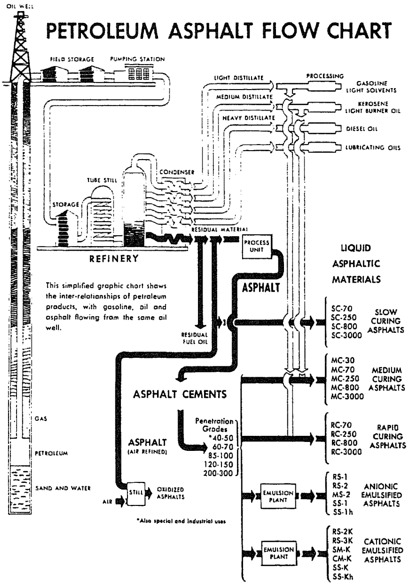


Figure 3–1. Simplified flow chart showing recovery and refining of petroleum asphaltic materials.

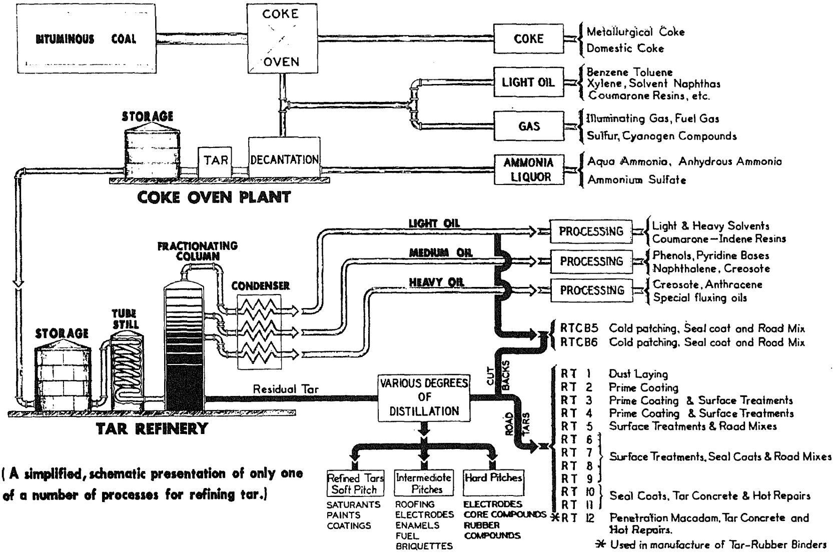


Figure 3–2. Simplified flow chart showing production of road tars from bituminous coals.

3–4. Asphalts

Asphalts may be natural or manufactured, and may be solid, semisolid or liquid in consistency. Natural asphalts occur in lakes (as lake asphalt), pits, or rock structures (as rock asphalt). Asphalt cement is one of the components from the refining of crude petroleum.

a. The military engineer is only secondarily concerned with the natural asphalts because they are not usually available in those areas in which the engineer is interested. He depends most generally upon the manufactured asphalts, which are obtained when crude petroleum is refined for the purpose of separating the various fractions (fig. 3–1). The crude oil vapors are separated into gasoline, kerosene, and fuel oils, while the residue is asphalt cement and lubricating oils. The longer the process and the higher the temperatures, the harder the residue becomes because of the increased loss in volatiles. All asphalt cements are solid or semisolid at room temperature (77° F.) and must be converted to a fluid state by heating.

b. Relative hardness is measured by the penetration test (para 3–16). The test determines the distance that a standard needle under a standard loading (weight) will penetrate a sample in a given time under known temperature conditions. From this test, a penetration number is assigned to the asphalt cement. The current Corps of Engineers specifications cover five paving grades as determined by this test. Each grade is designated by a penetration grade or AP number. (AP refers to asphalt manufactured from crude petroleum.) Table 3–1 lists the ranges presently recognized. Asphalt petroleum (AP) numbers corresponding to the grades are also listed.

Table 3–1. Penetration Grades and AP Numbers of Asphalt Cement

Penetration grade	*AP No.*	*Relative consistency*
40–50	7	Hard
60–70	5	
85–100	3	Medium
120–150	1	Soft
200–300	00	

c. When heating equipment is not available or is impractical to use, asphalt cement can be made fluid by adding solvents (called *cutterstock* or *flux oils*). Cutterstock may be any one of the more volatile petroleum distillate products. The resulting combination is called *asphalt cutback*. Exposure to air causes the solvents to evaporate and leave the asphalt cement to perform its functions.

(1) The classification of the cutback is based on the rate of evaporation of the distillate that is in the mixture. Gasoline or naphtha (highly volatile) will produce a *rapid curing cutback* (RC); kerosene (medium volatility) a *medium curing cutback* (MC); and a fuel oil (low volatility), a *slow curing cutback* (SC). At times, reference will be found to *road oils*, which are one of the SC grades of liquid asphalt, or in effect, a heavy petroleum oil. Table 3-2 shows the percentage of components by grade for the three types of asphalt cutbacks.

(2) As more cutterstock is mixed with a given amount of asphalt cement, a thinner liquid results. In practice, different amounts of cutterstock are added to a given amount of asphalt cement to obtain various viscosities, or grades, of cutbacks. The Corps of Engineers has approved a new set of specifications for cutbacks based on kinematic viscosity. The number assigned to each grade corresponds to the lower limit of kinematic viscosity as determined by a standard test. The upper limit of each grade is equal to twice the lower limit or grade number. The units used in the rest are *stokes* or *centistokes* $\left(\frac{\text{stokes}}{100}\right)$. Thus, a number 70 cutback refers to a viscosity range of 70 to 140 centistokes. The other grades and their limits are 250 (250–500), 800 (800–1600), and 3000 (3000–6000); in addition, the MC has a 30 grade. Figure 3-3 shows the scale of visconsity grades; the higher the viscosity, the thicker the liquid.

(3) Asphaltic penetrative soil binder is a special cutback asphalt composed of low penetration grade asphalt and a solvent blend of kerosene and naphtha. It is similar in character to a standard low viscosity, medium-curing cutback asphalt, but differs in many specific properties. It is used as a soil binder and dust palliative.

d. Emulsified asphalt is an emulsion of asphalt and water in which soaps, colloidal clays, and numerous other organic agents are used as emulsifiers. It is a heterogeneous system in which water forms the continuous phase of the emulsion and the minute globules of asphalt form the discontinuous phase. There is also an inverted asphalt emulsion in which the continuous phase is asphalt (generally liquid asphalt) and the discontinuous phase is minute globules of water in relatively small quantities. Emulsified asphalts may be anionic (negatively-charged asphalt globules) or cationic (positively-charged asphalt globules) depending upon the emulsifying agent.

(1) Emulsions are classified according to setting time. Setting or "breaking" rate is the speed with which the emulsion breaks or separates the asphalt and water. This rate is generally dependent upon the emulsifier used and the proportion of water to asphalt. Emulsions are described as *rapid setting* (RS), *medium setting* (MS), and *slow setting* (SS), and also by viscosity numbers (fig. 3-3).

(2) Emulsions are also grouped according to their ability to mix with damp aggregate. The RS emulsion breaks so fast that it cannot be mixed and is therefore called a *non-mixing* type. The MS and SS emulsions break slowly enough to permit good mixing until each particle of the aggregate is uniformly coated. Emulsions may also be satisfactorily used as a tack coat for bituminous pavements.

Table 3-2. Asphalt Cutback Composition (Expressed in Percent of Total Volume.)

Type	Components		Grades				
		Solvent	30	70	250	800	3000
Rapid curing RC	Asphalt cement		--------	65	75	83	87
		Gasoline or naphtha	--------	35	25	17	13
Medium curing MC	Asphalt cement		54	64	74	82	86
		Kerosene	46	36	26	18	14
Slow curing SC	Asphalt cement		--------	50	60	70	80
		Fuel oil	--------	50	40	30	20

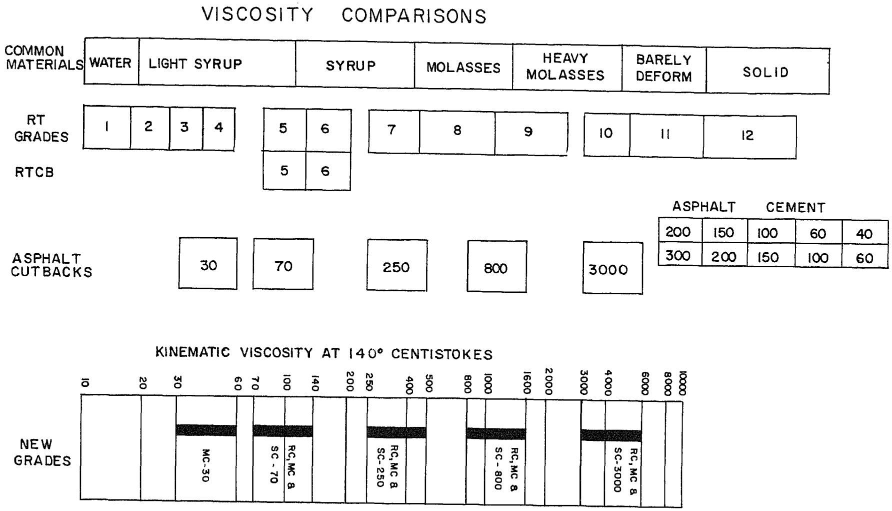


Figure 3–3. Viscosity grades at room temperature.

3–5. Tars

Tars are a product of the distillation of coal. No natural source of tars exists. *Coal tar* is a general term applied to all varieties of tar obtained from coal. It is produced by one of several methods depending on the desired end product. When bituminous coal is destructively distilled, coke and gas are formed, and tar, ammonia, light oils, sulfur, and phenol may be recovered. Coke-oven tar is produced in the greatest amount and its chemical, physical, and adhesive characteristics make it most suitable for road-tar purposes. Water-gas tar is obtained in the manufacture of carbureted water gas. The nature of the carbureting oil largely determines the character of the water-gas tar produced, and may vary widely in specific gravities, viscosities, and other physical and chemical properties.

a. Road tars are manufactured in twelve grades of viscosity (figs. 3–2 and 3–3). There are also some special grades for use in rubberized-tar binders. Grades 1 through 7 are liquid at room temperature, while grades 8 through 12 are semisolid or solid. The difference is due to the liquid coal distillates in the tar; the more distillate, the more liquid (or less solid) the tar. The road tar cutbacks (RTCB) are the products of cutting back the heavier or harder grades with coal tar distillates. Road tar cutbacks are manufactured in two viscosity grades (5 and 6) only.

b. Tar, which is insoluble in petroleum distillates, is sometimes mixed with oil-resistant, unvulcanized rubber to form a rubberized-tar binder material.

3–6. Characteristics and Uses of Bitumens

Tables 3–3 and 3–4 list the bituminous materials, sources, curing, temperatures, and grades associated with bituminous operations.

a. Safety Precautions. Extreme caution must be observed when handling bituminous materials. Asphalt cement, which is solid at room temperature, is heated to high temperatures to make it workable as a binder material. *Heated asphalt can cause severe burns if allowed to come in contact with the skin.* Asphalt cutbacks contain highly flammable volatiles. The vapors will ignite at relatively low temperatures, and the lowest temperature at which they will ignite is called the *flashpoint.* Minimum flashpoint for RC–250, RC–800 and RC–3000 is 80° F.; for MC–30 and MC–70, about 100° F.; for MC–250 to 3000 and SC–70, about 150° F., and for the remaining SC grades, about 25° F. higher per grade up to 225° F. for SC–3000. Note that the spraying and mixing temperatures (table 3–3) in many cases are above the flashpoint. Use extreme caution when handling these mixtures. *Do not expose their vapors to an open flame.*

b. Advantages and Disadvantages. Advantages and disadvantages of the bituminous materials used in construction are:

(1) Asphalt cement cutbacks are flammable. Also, all asphalts can be dissolved by petroleum products which may be spilled on them, such as durin refueling at an airfield. Tars, on the other hand are not affected by petroleum derivatives since the are products of coal. Tars, when used as a prime fo base courses, also seem to possess better penetratio qualities than asphalts. Tars are very temperatur susceptible, having a wide range in viscosity wit normal ambient temperature changes. Tar can become so soft during warmer weather that the pavement will rut under traffic. In colder weather, i can become so brittle that the pavement will crack

(2) Asphalt emulsions are not flammable an are liquid at normal temperatures. Since they ar mixed with water, they can be used with a more dam aggregate than required for the cutbacks. Additiona water may be added to the emulsion up to proportions of 1:3 for use in slurry seal coats. Becaus emulsions contain water they have certain disadvantages. During freezing weather, the emulsion can freeze and the components separate. Also, emulsions are difficult to store for extended periods, because they tend to "break" even in unopened drum In shipping emulsions, the water in the mixtur takes up valuable space which could be used to transport hard-to-obtain materials.

Table 3-3. Characteristics of Bituminous Materials

Material	Form	Grade designation	Temperature of application ranges				Flashpoint (min)		Remarks
			Spraying[1]		Mixing				
			° F.	° C.	° F.	° C.	° F.	° C.	
Penetrative Soil Binder	Liquid	–	130–150	55–65	–	–	80	27	Contains naphtha. Caution: Highly flammable.
Cutback (RC)	Liquids—asphalt residues fluxed with more volatile petroleum distillate.	RC–70	[2] 105–175	[2] 41–79	95–135	35–57			Rapid curing cutbacks contain highly volatile naphtha cutterstock. Naphtha evaporates quickly leaving asphalt cement binder permitting early use of surface. Caution: Highly flammable.
		–250	145–220	63–104	135–175	57–79	80	27	
		–800	180–255	[2] 82–124	170–210	77–99	80	27	
		–3000	[2] 215–290	[2] 102–143	200–240	93–116	80	27	
(MC)	do	MC–30	70–140	21–60	55–95	13–35	100	37	Medium curing cutbacks contain less volatile kerosene cutterstock. Kerosene evaporates less rapidly than naphtha. Caution: Flammable.
		–70	105–175	41–79	95–135	35–57	100	37	
		–250	145–220	63–104	135–175	57–79	150	65	
		–800	180–255	82–124	170–210	77–99	150	65	
		–3000	215–290	102–143	200–240	93–116	150	65	
(SC)	do	SC–70	105–175	41–79	95–135	35–57	150	65	Slow curing cutbacks contain slightly volatile diesel fuel cutterstock. Diesel fuel evaporates slowly. Caution: Flammable.
		–250	145–220	63–104	135–175	57–79	175+	79+	
		–800	180–255	82–124	170–210	71–99	200+	93+	
		–3000	215–290	102–143	200–240	93–116	225+	107+	
Asphalt Cements (AC).	Solids	40–50	------	------	300–350	149–177	------	------	Penetrations 40 to 100 used for crack and joint fillers. Penetrations 70 to 300 used for plant mixes, pentration macadam, and surface treatment. Use test (TM 5–530) to determine flashpoint.
		60–70	285–350	141–177	275–325	135–163	------	------	
		85–100	285–350	141–177	275–325	135–163	------	------	
		120–150	285–350	141–177	275–325	135–163	------	------	
		200–300	260–325	127–163	200–275	93–135	------	------	

Powdered Asphalt (PA).	Hard and solid asphalts ground to powder.	-----	-----	-----	-----	-----	-----	-----	Used with SC to produce extra tough road surfaces.
Asphalt Emulsions (RS).	Liquids—asphalt particles held in an aqueous suspension by an emulsifying agent.	RS-1	50-140	10-60	Nonmixing	10-60	-----	-----	Freezing destroys emulsion. Use for road and plant mixes with coarse aggregates (SS). All emulsions with "K" suffix are cationic.
		RS-2	50-140	10-60	50-140	10-60	-----	-----	
		RS-2K	50-140	10-60	50-140	10-60	-----	-----	
		RS-3K	50-140	10-60	50-140	10-60	-----	-----	
(MS)	do	MS-2	50-140	10-60	50-140	10-60	-----	-----	
		SM-K	50-140	10-60	50-140	10-60	-----	-----	
		CM-K	50-140	10-60	50-140	10-60	-----	-----	
(SS)	do	SS-1	50-140	10-60	50-140	10-60	-----	-----	
		SS-1h	50-140	10-60	50-140	10-60	-----	-----	
		SS-K	50-140	10-60	50-140	10-60	-----	-----	
		SS-Kh	50-140	10-60	50-140	10-60	-----	-----	
Road Tars (RT)	Liquids	RT-1	60-125	15-52	-----	-----	-----	-----	Priming oils. RT-4 through RT-12 not generally used.
		RT-2	60-125	16-52	-----	-----	-----	-----	
		RT-3	80-150	27-66	-----	-----	-----	-----	
Road Tar Cutbacks (RTCB).	do	RTCB-5	60-120	16-49	-----	-----	-----	-----	Patching mixtures. Caution: Flammable.
		RTCB-6	60-120	16-49	-----	-----	-----	-----	
Rock Asphalt	Solids	-----	-----	-----	-----	-----	-----	-----	Mixed and used locally where found. Cutback may be added if necessary.

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[1] Low temperature is based on a viscosity of 200 centistokes kinematic viscosity and the higher temperature is based on a 50 centistokes viscosity.

[2] RC Cutbacks are seldom used for spraying.

Table 3-4. Typical Uses of Bituminous Materials[1]

Purpose or use	Grade or designation		
	CB - Asphalt cutback[2]		
	RC Rapid curing	MC Medium curing	SC Slow curing
Dust palliative	----- DCA-70[3]	MC-30, 70, 250 A.P.S.B.[4]	SC-70, 250
Prime coat:			
Tightly bonded surfaces	-----	MC-30	-----
Loosely bonded - fine grained surfaces.	-----	MC-70	SC-70
Loosely bonded - coarse grained surfaces.	-----	MC-250	SC-250
Tack coat	RC-250, 800	MC-250, 800	-----
Surface treatment and seal coat:			
Coarse sand cover	RC-70, 250	MC-250, 800	-----
Clean coarse aggregate cover	RC-250, 800, 3000	MC-800	-----
Graded gravel aggregate cover	-----	MC-250, 800	SC-800
Gravel mulch	-----	MC-250	SC-250
Mixed in-place - Roadmix:			
Open-graded aggregate:			
Sand	RC-70, 250	MC-800	-----
Maximum diameter 1 in., high percentage passing No. 10.	-----	MC-800	-----
Macadam aggregate	RC-250, 800	-----	-----
Dense-graded aggregate:			
High percentage passing No. 200	-----	MC-250	SC-250
Maximum diameter 1 in., medium percentage passing No. 200.	-----	MC-250, 800	SC-250, 800
Premix or cold patch:			
Open-graded aggregate	RC-250	MC-800	SC-800
Dense-graded aggregate	-----	MC-250	SC-250
Cold-laid plant mix:			
Open-graded aggregate:			
Sand	RC-250, 800	-----	-----
Maximum diameter 1 in., high percentage passing No. 10.	RC-800	-----	SC-800
Macadam aggregate	RC-800, 3000	-----	-----
Dense-graded aggregate:			
High percentage passing No. 200	-----	MC-800	SC-800
Maximum diameter 1 in., medium percentage passing No. 200.	-----	MC-800	SC-800
Aggregate precoating followed with asphalt	-----	MC-30	SC-70
Hot-laid plant mix	RC-3000	MC-3000	SC-3000
Penetration macadam:			
Cold weather	RC-800, 3000	-----	SC-3000
Hot weather		-----	-----

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[1] Prevailing temperature during construction also affects selection of bitumen and may be the determining factor rather than size and gradation of aggregate.

[2] Caution: Do not overheat aggregate when cutbacks are used to produce hot mixes.

[3] DCA-70 is a water emulsion of a polyvinyl acetate containing chemical modifiers (formerly UCAR-131). Proprietary product of Union Carbide Corporation, New York, N.Y.

[4] Asphaltic penetrative soil binder.

Table 3-4. Typical Uses of Bituminous Materials — Continued

AC Asphalt cement with a penetration of -	AE Anionic and cationic asphalt emulsion	RT-RTCB Road tar and road tar cutback
-----------------------	SS-1, 1h -----------	RT-1 ---------------
-----------------------	--------------------	RT-2 ---------------
-----------------------	--------------------	RT-3 ---------------
-----------------------	---------------------	RT-4 ---------------
200-300 ---------------	RS-1, 2 -------------	RT-4, 5, 6, 7, 8, 9
-----------------------	RS-1, 2 -------------	
120-150, 200-300 ------	---------------------	RT-6, 7, 8, 9, 10
-----------------------	---------------------	
-----------------------	---------------------	
-----------------------	---------------------	RT-6 ---------------
-----------------------	MS-2 ----------------	
85-100 ----------------	MS-2 ----------------	RT-7 ---------------
-----------------------	SS-1h ---------------	RT-5, 6, 7 ---------
-----------------------	SS-1 ----------------	RT-6, 7, 8, 9
-----------------------	MS-2 ----------------	RT-5, 6, 7, 8 or
-----------------------	---------------------	RTCB-5, 6 ----------
-----------------------	MS-2 ----------------	
-----------------------	MS-2 ----------------	
-----------------------	---------------------	
-----------------------	SS-1 ----------------	RT-5, 6, 7, 8, 9 ---
-----------------------	SS-1 ----------------	
-----------------------	---------------------	
85-100, 120-150 -------	---------------------	RT-11, 12 ----------
120-150 ---------------	RS-1 ----------------	RT-10, 11 ----------
85-100 ----------------	RS-1 ----------------	RT-12 --------------

3–7. Representative Samples

a. The materials used in conducting tests must be representative of those that will be used in construction. Otherwise, test results will be entirely misleading. Sampling of materials for testing should receive close attention. Even when large samples must be subdivided into small units for the actual tests, care must be taken to keep the sample representative of the original mass. Aggregate samples should be quartered (para 2–22) to maintain a representative fraction of the larger unit until the proper size sample is obtained.

b. Methods for sampling natural deposits of sands and gravels are discussed in Section II of Chapter 2 and are applicable here.

c. Samples of bituminous materials may be taken at the place of manufacture or at the point of delivery, as expedient. For the purposes of this manual, it is assumed that sampling is done at the point of delivery. Samples may be taken for either of two purposes; to obtain an average of the delivered material as nearly as possible and to ascertain the maximum variation in the material's characteristics. Samples may be taken for analysis to identify bituminous materials if records are not available.

3–8. Aggregate Sampling

Aggregate varies in size from the larger stones or rocks to the gravels and sands. These materials for paving may still be in their natural deposits or may be in stockpiles previously gathered.

a. Stone From Ledges and Quarries. Separate samples of unweathered stone weighing at least 50 pounds each should be obtained from all strata that appear to vary in color and structure. A sketch, plan, and elevation, showing the thickness, length, width, and location of the different layers, should be prepared so that the quantity available can be estimated.

b. Natural Deposits of Sand and Gravel. Samples should be selected which represent the different materials available in the deposit. The area should be sketched and the approximate quantities of different materials indicated. If the deposit is an open-face bank or pit, the sample should be taken by channeling the face, so that it will represent material that visual inspection indicates may be used. The face should be cut immediately before sampling and any material which has fallen from the surface along the face discarded. Any overlying material (overburden) that is not suitable for use as an aggregate is not included in the sample, since this material would be stripped away when the aggregate is removed from the pit. It probably will be necessary to make test borings or dig test pits to determine the approximate extent of the material. If test pits are dug, they must be adequately shored to prevent material from caving in on personnel working in the pit. Representative samples for each change in strata can be obtained from the pit. If the material being sampled is all sand, about 25 pounds is sufficient for the tests. If it consists of sand and gravel, a somewhat larger sample (about 100 lb) will be required for preliminary tests; the coarser the gravel portion, the larger the sample should be.

c. Stockpiles of Sand and Gravel. If the material has been stockpiled previously, great care must be exercised in obtaining samples, since there is a natural tendency toward segregation, that is, separation of similar size fractions into groups in the stockpile. The material near the outer edges and near the base of the pile is likely to be coarser than the average. A face should be cut into the stockpile near the base, near the center, and near the top, on at least two opposite sides. Samples obtained from these different sections of the pile are combined to give a representative sample.

d. Commercial Aggregates. It is preferable to obtain samples of commercial aggregates at the plant during loading from stockpiles or bins when this is possible. Separate samples should be obtained at different times while the material is being loaded, in order to determine variations in the grading of the material. Samples from a bin should be taken from the entire cross section of the flow of material as it is being discharged. Testing separate samples will give a better idea of variations that occur, but samples should be mixed and reduced by quartering when the average condition is desired. When it is not practicable to visit the plant to obtain samples, the next preferred method is to sample the material in cars or trucks, or while it is being unloaded. Samples from railroad cars should be taken from three or more trenches dug across the car at points that appear on the surface to be representative of the material. The person obtaining the sample should always bear in mind that segregation of the different sizes has probably taken place, and samples which are representative should be chosen.

3–9. Bituminous Materials Sampling

a. Clean, dry containers must be used in sampling and the containers must be tightly closed and properly marked. A sample for routine examination should not be less than 1 quart.

b. When sampling liquid bituminous materials from storage tanks or tank cars, samples should be obtained from near the top, middle, and bottom. Semisolid and solid materials shipped in tank cars may be made fluid by the application of heat and sampled in the same way. Samples can be taken from drain cocks on the side of the tank or car. If cocks are not present, the samples can be obtained by lowering weighted bottles or cans into the material, as illustrated in figure 3-4. The bottle or can should be fitted with a stopper which can be removed by a string or wire after it has been lowered to the proper depth. The samples from the three levels should be tested separately to detect stratification.

c. In sampling semisolid or solid materials in drums, barrels, or cakes, samples should be taken at least 3 inches below the surface and 3 inches from the side of the container or cake. A clean hatchet may be used on hard material and a stiff putty knife on soft material. An auger, brace with a ¾ inch or larger bit, or other suitable tools may also be used.

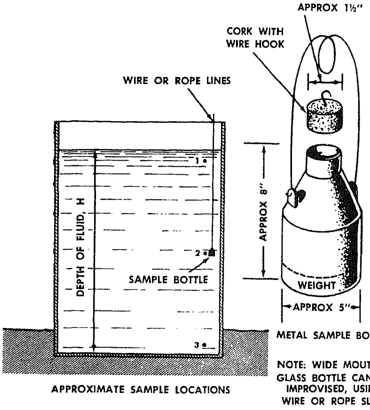


Figure 3-4. Sampling liquid bituminous materials from
car or storage tank.

Section III. FIELD IDENTIFICATION

3-10. Introduction

Tests which would be conducted on bituminous materials in a large laboratory, usually for the purpose of checking compliance with established specifications, are not normally considered field tests but are described for information. The field tests which are discussed are limited to the simple identification procedures, flashpoint tests, and the penetration test. These field identification procedures are applicable to both tars and asphalts. They are intended not so much for determining the exact specifications to which a material conforms, but for determining for what it can be used and how it can be used safely.

a. Bituminous materials are often found stockpiled in unmarked or incorrectly marked containers. This leads to confusion and delay in construction since the various types and grades of bituminous material are manufactured for a specific purpose.

b. Field identification is important to the military engineer and this importance can be summarized in the following way:

(1) Knowing the type and grade of the material, the type of surface that can be constructed may be determined.

(2) With the type of surface known, the construction procedure may be outlined and scheduled for a particular target date.

(3) The procedure will, in turn, determine the proper equipment for the job.

(4) The safety procedures are also dependent upon the identified material.

c. The aggregate must also be tested to deter
its acceptability for the bituminous constructio

3-11. Bitumen Identification

Field tests may be performed to identify the bit
nous paving materials as asphalt cement, asphalt
back, asphalt emulsion, road tar, or road tar cutb
In addition, it is necessary to identify as closel
possible the viscosity grade of the bitumen (fig. 3
In order to distinguish among the several asph
and tar products, it is necessary to know somet
of their origin, physical properties, and the ma
in which they are normally used. Some of this in
mation is contained in tables 3-3 and 3-4. The i
tification procedure outlined in figure 3-5 is b
upon a consideration of the physical propertie
these materials.

a. Asphalts and Tars. The first procedure in
identification of an unknown bituminous materi
to determine whether it is an asphalt or a tar. T
may be differentiated by a solubility test. To
form the test, simply attempt to dissolve an unkn
sample (a few drops, if liquid, or enough to c
the head of a nail, if solid) in any petroleum d
late. Kerosene, gasoline, diesel oil, or jet fuel is
able for this test. One or more of these is usu
available to the engineer in the field. Since aspha
derived from petroleum, it will dissolve in the
troleum distillate. Road tar will not dissolve. If
sample is an asphalt, the sample-distillate mix
consist of a dark, uniform liquid. Asphalt cem

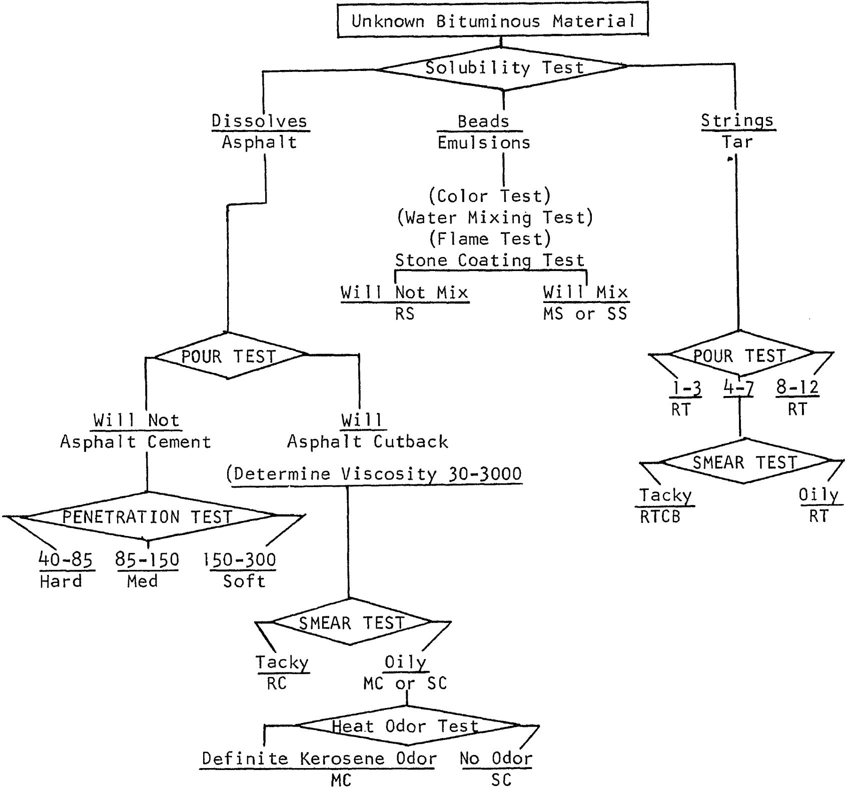


Figure 3–5. Identification of unknown bituminous materials.

or cutbacks dissolve completely; the asphalt in emulsions dissolves and forms black beads or globules in the bottom of the container. If it is a road tar, sample will be a dark, stringy, undissolved mass in the distillate. A check can be made by spotting a piece of paper or cloth with the mix. Asphalt dissolved in distillate will produce a brown to black spot or stain; the clear distillate above the settled tar will not cause a stain. The solubility test provides a positive method of identification.

b. Cements and Cutbacks. The various grades of asphalt cement are solid at room temperatures while cutbacks are liquid. Hence, a pour test will distinguish between them. Place a sample of the material in a small container and attempt to pour it. If the material does not "pour", it is an asphalt cement. If it pours, it is a cutback or emulsion. Note that at 77° F., even the softest asphalt cement will not pour or deform if the container is tilted.

c. Grades of Asphalt Cement. The various grades of asphalt cement are distinguished principally by their hardness, as measured by a field penetration test. This information may be sufficient for planning, or in some cases actually starting emergency construction. The exact penetration grade is not determined, but the field test will distinguish between hard, medium, or soft groups of asphalt cement. This test is performed by attempting to push a sharpened pencil or nail into the container of asphalt (at about 77° F.) using a firm strong force. By "strong" is meant perhaps a 10 pound force. If only a slight penetration is made with considerable difficulty, a hard asphalt cement is present. If the penetration is made without great difficulty—slowly, perhaps, but positively, and to any desired depth, probably a medium asphalt cement is present. If the penetration is made with ease, the asphalt cement is naturally in the high penetration scale, a soft asphalt cement.

Even the highest penetration grade will not pour or deform at 77° F. if its container is tilted.

d. Asphaltic Cutbacks. As stated previously, an asphalt cement would not pour at 77° F. but a cutback would. Therefore, the pour test can be utilized to determine whether or not the unknown material is an asphalt cutback. If the material pours, it is an asphalt cutback. The approximate viscosity grade number of the cutback is found by comparing the flow to well-known materials such as water, syrup, and others. In this test, if made at a temperature below 77° F., the materials will appear more viscous than at 77° F. and the opposite if pour-tested when warmer than 77° F. It has been found that the cutbacks of a given viscosity grade will pour in a manner similar to the following:

30's—Water
70's—Light Syrup
250's—Syrup
800's—Molasses
3000's—Barely Deform

After the pour test, the approximate viscosity grade of the cutback is known, but the actual type (RC, MC, SC) is not.

(1) To determine if a cutback is an RC or not, the "smear test" is performed. This is done by making a uniform smear of the substance on a piece of glazed paper or other convenient nonabsorbent surface. This will give volatile materials, if present, a chance to evaporate. Since RC's are cut back with a very volatile substance, most of the volatiles will evaporate within 10 minutes. The surface of the smear will then become extremely tacky. This is not true of the lighter grades of MC's and SC's, which remain fluid and smooth for some time. An MC will not result in a tacky surface for a matter of hours; for SC materials, several days may be required.

(2) To identify an 800 or 3000 grade MC or SC cutback, a "prolonged" smear test is used. This is necessary because these grades of MC's and SC's contain such small quantities of cutterstock that they, too, may become tacky in the 10-minute period specified above. A thin smear of the material is made on a nonabsorbent surface and left to cure for at least 2 hours. By the end of that time, if the material being tested is an RC, the smear will have cured and will be hard, or just slightly sticky. However, if the material being tested is an MC or SC, the smear will be uncured and still quite sticky. If the material is an RC 3000, it will cure completely in 3 hours, whereas an RC 800 will take about 6 hours to cure. Even after 24 hours, an MC or SC will still be sticky.

(3) Since MC's are cut back with kerosene and SC's with oil, this fact may be employed to differentiate between them. Heat is used to drive off the kerosenes, if present, and result in an odor from the vapors. It is best to heat the unknown sample in a closed container in order to capture the escaping vapors, being careful not to apply too much heat. If the sample is an MC, it will have a strong petroleum or kerosene odor. On the other hand, if the sample is an SC, no kerosene or petroleum odor will be detected, even though it might smell a little like hot motor oil. The ability to tell an RC from an MC, and an SC from either, is perhaps as important as any other part of field identification.

e. Asphalt Emulsions. Another asphaltic material used in paving work is asphalt emulsion, which is a mixture of asphalt, water, and an emulsifying agent. It presents no problem in identification, since it is usually distinguished by its dark brown color, while the other bitumens are black. If mixed with kerosene or some other petroleum distillate, the emulsion can be detected by the appearance of small black globules or beads which fall to the bottom of the container. If mixed with water, an emulsion will accept the extra water and still remain a uniform liquid. The other bitumens will not mix with water. Since an emulsion contains water, a small piece of cloth saturated with it will not burn if a flame is applied. The other bitumens will burn or flame. After it has been established that the material is an emulsion, it is still important to know whether or not the emulsion is a mixing grade. To tell if the emulsion is a mixing grade (slow-setting, MS or SS) or a nonmixing grade (fast-setting, RS), the best way is to try to mix a small amount (6 to 8 percent by weight) with damp sand using a metal spoon. A fast-setting emulsion cannot be mixed; it breaks immediately, gumming up the spoon with the relatively hard original asphalt cement. A slow-setting emulsion mixes nicely, coating the sand. During this test, not too much emulsion should be added to the sand. This would saturate the sand and not give conclusive results. No further identification is necessary since both MS and SS grades are largely used for the same jobs.

f. Road Tars. If the unknown bituminous material did not dissolve in the solubility test, but formed a stringy mass (fig. 3-5), the material is a tar. The next step is to determine its viscosity grade by the pour test. By comparing the flow to that of the common materials (fig. 3-3), the viscosity of the tar may be closely estimated. The grades vary from RT-1 to RT-12. If the identified tar has a viscosity in the range of an RT-4 to RT-7 material, a smear test must be performed to determine whether it is a road tar or a road tar cutback. The smear test is performed in the manner previously described for cutback asphalt (*d* (1) above). A great increase in stickiness in about 10 minutes identifies a road tar cutback. No apparent change in consistency after 10 minutes shows the material to be a road tar. It is of no particular importance which grade of cutback is available, since both are used under approximately the same conditions.

3–12. Aggregate Identification

Available aggregate may not always have all the characteristics that are desirable. The one meeting most of the requirements is usually selected, if not rejected for reasons such as availablity, length of haul, or overburden.

a. Angular and Rough. The aggregate in a pavement must transmit the traffic load to the base, usually by the interlocking and surface friction of the different particles. Angular particles with a rough texture are the best for this purpose since they do not tend to slide past each other. However, more binder may be required since the angular shape has a greater surface area per unit volume than a round particle.

b. Hard and Durable. The aggregate must withstand the loads without cracking or being crushed. Resistance to weathering is also a function of the durability. The resistance-to-wear of an aggregate can be determined by the Los Angeles Abrasion Test if the equipment is available. The equipment and procedures are detailed in the AASHO method T96 and in ASTM Designation C 131. The equipment for the above test is usually not available for field testing and the Mohs hardness scale may be used to determine the hardness of the aggregate. This scale is fully explained in TM 5–545. It ranges from 1 for talc or mica to 10 for diamond. By trying to scratch the aggregate or the common materials and vice versa, it is possible to establish which is harder, and from this, the hardness of the aggregate. If both scratch each other, the hardness of both is the same. Be sure to rub the "scratch" mark to see that it is really a scratch and not a powdering of the softer material. Some common materials and their hardness are: fingernail about 2, copper coin between 3 and 4, knife blade about 5, and window glass about 5.5.

c. Clean and Dry. The bituminous binder must penetrate in to the pores of the aggregate, and also adhere to the surface of the particles. Aggregates coated with clay or dust or having water-filled pores will prevent the penetration or the adherence of bitumen and result in stripping of the binder. If the aggregate is not clean, it should be washed either as part of the crushing operation or by spreading it on a hard surface and hosing it with water. When washing is impracticable, dry screening may remove a great deal of dust and clay. Even hand picking may have to be done if no other method can be used. The aggregate should be made as clean as possible with the equipment and manpower available.

d. Impermeable. Affinity for water can make an aggregate undesirable. If the aggregate is porous and absorbs water easily, the binder can be forced out of the pores, the bond between the aggregate and binder weakens and breaks, and "stripping" occurs. Stripping is the loss of bituminous coating from the aggregate particles due to the action of water, leaving exposed aggregate surfaces. One of the three following tests can be used to determine the detrimental effect of water on a bituminous mix:

(1) *Stripping test.* A test sample is prepared by coating a specific amount of aggregate with bituminous material at the applicable temperature for the grade of bitumen to be used. The mixture is spread in a loose, thin layer and air-cured for 24 hours. A representative sample is placed in a jar (up to no more than one-half of its capacity) and covered with water. The jar is closed tightly, and allowed to stand 24 hours. At the end of 24 hours, the jar with the sample is shaken vigorously for 15 minutes. A visual examination is made to determine the percentage of exposed aggregate surface which is then reported as percent stripping.

(2) *Swell test.* Asphaltic mixtures containing fines of doubtful quality are sometimes measured for swell as a basis for judging the possible effects on a pavement. This test is more frequently used with dense-graded mixtures using liquid asphalts. A sample of the mix is compacted in a metal cylinder and cooled to room temperature. The specimen and mold are placed in a pan of waetr and a dial gage is mounted above the sample in contact with the surface. An initial reading is taken. The specimen is allowed to soak for a specified period (usually 24 hours) or until there is no further swelling. Another reading of the dial is taken. The difference in readings is the swell of the mixture. Experience has shown that bituminous pavement made with clear, sound stone, slag, or gravel aggregate and mineral filler produced from limestone will show test values of swell of less than 1.5 percent of the original specimen thickness. Aggregates of doubtful character should be tested for conformance to specifications of test method AASHO T–101.

(3) *Immersion-compression test.* This test is intended to measure the loss of Marshall stability resulting from the action of water on compacted bituminous mixtures containing penetration-grade asphalt. The result is a numerical index of reduced stability obtained by comparing the Marshall stability (para 3–24) with the stability of specimens that have been immersed in water for a prescribed period. Eight standard Marshall specimens (4 in. in diameter and $2\frac{1}{2} \pm \frac{1}{16}$ in. high) are prepared for each test. The specific gravity of each of the specimens is determined. The set of eight are separated into two sets of four so that the average specific gravity of one set is essentially the same as the other set. One set is now tested using the Marshall method (para 3–24) and the other set is first immersed in water (at

140°±1° F.) for 24 hours and then tested. The result is computed as a ratio of soaked stability to unsoaked stability, expressed as a percentage, or

$$\text{Index of reduced stability} = \frac{S_2}{S_1} \times 100$$

Where:

S_1=average stability of unsoaked specimens
S_2=average stability of soaked specimens

Mixes with an index of less than 75 percent are rejected or an approved method of processing aggregate and treating asphalt is required to increase the index to a minimum of 75 percent.

e. Particle Size. In bituminous construction, it is common practice to designate aggregate according to particle size. There are three types of designations under this system based on two sieve sizes, No. 10 and No. 200.

(1) Coarse aggregate is all material too large to pass the No. 10 sieve.

(2) Fine aggregate passes the No. 10 sieve, but is retained on the No. 200 sieve. In bituminous paving, the fine aggregate is usually a sand, but small pieces of crushed rock may be used.

(3) Mineral filler or mineral dust refers to all nonplastic materials which pass the No. 200 sieve. Most clays are too plastic and are not used. Generally, crushed rock dust, agricultural mineral filler, lime, or portland cement may be used as the mineral filter.

f. Gradation.

(1) *Uniform gradation.* All particles are approximately the same size and normally less than 1 inch.

(2) *Macadam gradation.* This group consists of uniformly sized particles except that they are in excess of 1 inch.

(3) *Open gradation.* The sizes of particles in this group may have a considerable range from large to small, usually containing little or no mineral filler or in which the void spaces in the compacted aggregate are relatively large.

(4) *Dense gradation.* There is a good representation of all particle sizes, coarse, fine, and mineral filler.

Section IV. TESTS ON BITUMENS

3-13. Discussion

a. The field identification tests on bitumens (para 3-11) identify the material during expedient conditions or until such time as the more detailed tests can be performed. The identification determines whether the material is an asphalt or a tar, and whether it is a cutback or an emulsion.

b. Bituminous materials are manufactured to meet specifications established by the Federal government, American Association of State Highway Officials (AASHO), and American Society for Testing and Materials (ASTM). These specifications define the extreme limits permitted in the manufacture of the material and assure the user that the material will possess definite characteristics and fulfill the project requirements. Conformance to specifications tests include determining the specific gravity of the material, its solubility, its analysis by distillation, and its softening point. The equipment for performing these tests is not included in the asphalt test set, and is not normally available to the soil technician. However, these tests are described for information and when the equipment is available, to identify the material beyond field identification, to furnish information for mix design, or to establish safe handling procedures.

3-14. Specific Gravity Test of Bituminous Materials

Specific gravity of a bituminous material is defined as the ratio of the weight of a given volume of the material at 77° F. (25° C.) to the weight of an equal volume of distilled water at the same temperature. The results of a bitumen specific gravity test are used in the selection of the temperature-volume-weight correction factor to convert volumes to volume at 77° F. (25° C.). Space is provided on DD form 1216 (fig. 3-6) to make this determination. A pycnometer (specific gravity bottle) is weighed empty and reweighed filled with water. The water is emptied from the bottle and the bitumen is added. The pycnometer and bitumen are weighed. Water is added to the same level as at the start of the test and the entire combination is weighed. The procedure outlined on the form is followed to compute the weight of water displaced by the bitumen and the bitumen's apparent specific gravity. The specific gravity of asphalt cements will generally be in the range of 1.00 to 1.06 with the higher values being characteristic of the harder materials. For details of the specific gravity test, see ASTM Designation D 70.

SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS

DATE 2 APR 1970

PROJECT QUANG HO BASE

JOB RUNWAY NO. 1

COARSE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING 1" SIEVE AND RETAINED ON 16 SIEVE				
SAMPLE NUMBER	CA			
1. WEIGHT OF OVEN - DRY AGGREGATE	378.3			
2. WEIGHT OF SATURATED AGGREGATE IN WATER	241.0			
3. DIFFERENCE *(Line 1 minus 2)*	137.3			
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line 1})}{(\text{Line 3})}$	$\frac{378.3}{137.3} = 2.755$			
FINE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING NUMBER 3/8" SIEVE				
SAMPLE NUMBER	FRBS			
4. WEIGHT OF OVEN - DRY MATERIAL	478.8			
5. WEIGHT OF FLASK FILLED WITH WATER AT 20° C	678.6			
6. SUM (Line 4 + 5)	1157.4			
7. WEIGHT OF FLASK + AGGREGATE + WATER AT 20° C,	977.4			
8. DIFFERENCE *(Line 6 minus 7)*	180.0			
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line 4})}{(\text{Line 8})}$	$\frac{478.8}{180.0} = 2.660$			
FILLER	**UNITS** *(Grams)*			
SAMPLE NUMBER	LSD			
9. WEIGHT OF OVEN - DRY MATERIAL	466.5			
10. WEIGHT OF FLASK FILLED WITH WATER AT 20° C,	676.1			
11. SUM *(Line 9 + 10)*	1142.6			
12. WEIGHT OF FLASK + AGGREGATE + WATER AT 20° C,	973.8			
13. DIFFERENCE *(Line 11 minus 12)*	168.8			
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line 9})}{(\text{Line 13})}$	$\frac{466.5}{168.8} = 2.762$			
BINDER	**UNITS** *(Grams)*			
SAMPLE NUMBER	6873			
14. WEIGHT OF PYCNOMETER FILLED WITH WATER	61.9595			
15. WEIGHT OF EMPTY PYCNOMETER	37.9215			
16. WEIGHT OF WATER *(Line 14 minus 15)*	24.0380			
17. WEIGHT OF PYCNOMETER + BINDER	47.8617			
18. WEIGHT OF BINDER *(Line 17 minus 15)*	9.9402			
19. WEIGHT OF PYCNOMETER + BINDER + WATER TO FILL PYCNOMETER	62.1568			
20. WEIGHT OF WATER TO FILL PYCNOMETER *(Line 19 minus 17)*	14.2951			
21. WEIGHT OF WATER DISPLACED BY BINDER *(Line 16 minus 20)*	9.7429			
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line 18})}{(\text{Line 21})}$	$\frac{9.9402}{9.7429} = 1.020$			

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*
Sp-4 A. McMurray	Sp-4 A. McMurray	Sgt. Arthur Price

DD FORM 1 DEC 65 1216 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

Figure 3-6. Data sheet, specific gravity of bituminous mix components.

3–15. Flashpoint Tests

These tests are applicable to asphaltic materials and are of some use in the identification of these materials. Their greatest usefulness, however, is in determining safe heating temperatures. Material heated above its flashpoint presents a very real danger, particularly if it is exposed to an open flame.

a. Cleveland Open Cup. This test is used for all petroleum products except fuel oils and those having open cup flashpoints below 175° F. (79° C.).

(1) *Apparatus.* The equipment consists of a flashpoint test kit (fig. 3–7), a knife, a frying pan or copper beaker, a hotplate and a stopwatch.

(2) *Procedure.* It is best to perform this test away from any bright light or to shield the apparatus, if necessary. The cup should be cleaned thoroughly before starting. The bituminous material may be heated until it is fluid enough to pour into the cup. Within the last 30° F., before the expected flashpoint is reached, the cup and its contents should not be moved or disturbed. Even air movement or drafts across the specimen surface should be prevented. The procedure of the test is as follows:

(*a*) Set up the open cup apparatus.

(*b*) Adjust the thermometer in a vertical position ¼ inch above the bottom of the cup and about midway between the center and back of the cup.

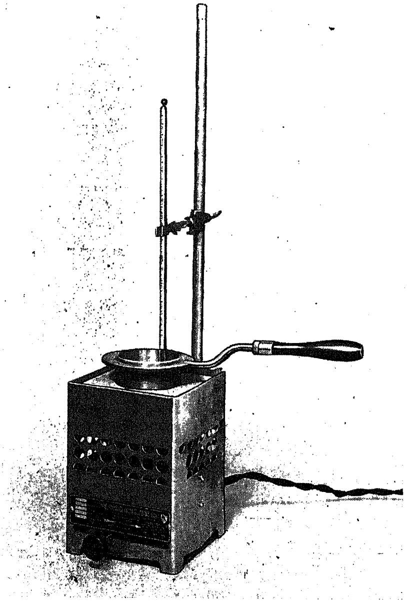


Figure 3–7. Flashpoint apparatus, Cleveland open cup.

(*c*) Fill the cup with the heated materi until the top of the meniscus is exactly at the fillin line, and let the material cool.

(*d*) Apply heat to the cup so that the spec men temperature is raised at a rate of 25° to 30° per minute until a temperature of about 100° below the probable flashpoint is reached.

(*e*) Reduce the heat at this time, and adju it so that for the last 50° F. before the expected flas point, the temperature rise will be not less than and not more than 11° F. per minute. Use the sto watch to regulate this rate. Failure to set the ra of rise between these limits will result in inaccura readings.

(*f*) Adjust a test flame to about 5/32 inc in diameter.

(*g*) Starting at least 50° F. (28° C.) belo the expected flashpoint, pass the test flame in straight line across the center of the cup at rig angles to the thermometer and level to the upp edge of the cup. The time for each pass should be n more than 1 second.

(*h*) Repeat the test flame pass for each su cessive 5° F. (2.8° C.).

(3) *Results.* The flashpoint is reached when flash (distinct flicker) appears at any point on t surface of the material. Read the thermometer this time and record the temperature as the flas point.

(4) *Repeatability.* Duplicate tests on the san material should not differ by more than 15° (8.3° C.).

b. Tag Open Cup. This test is used for the RC an MC asphalt cutbacks having a flashpoint belo 175° F. (para 3–6*a*).

(1) *Apparatus.* The equipment for this te consists of a flashpoint tester (Tag open cup (fi 3–8)), a hotplate, a thermometer (20° to 230° F. 1° divisions), and a torch or test flame.

(2) *Procedure.* The tester should be set up in draft-free and dimly-lighted location. The coppe water bath is to be filled to ⅛ inch below the to (when the top is in place). The bath may have a overflow to control the water level. The glass cup cleaned and dried and the water bath assembled. Th procedure of the test is as follows:

(*a*) Place the thermometer vertically midwa between the center and the outer edge of the cu and diametrically across from the guide wire. Set th bottom of the bulb about ¼ inch above the bottom the glass cup.

(*b*) Fill the glass cup with the sample to ju 5⁄16 inch below the edge.

(*c*) Place the guide wire in position touchin the rim of the glass cup.

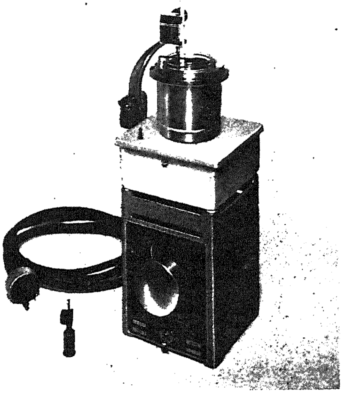


Figure 3-8. Flashpoint apparatus, Tag open cup.

(*d*) Adjust the heat for the sample temperature to rise at 2°±½° F. per minute. The thicker materials should be stirred occasionally.

(*e*) Adjust the test flame to form a 5/32-inch sphere.

(*f*) Remove any bubbles which may have formed on the surface before starting the flame test.

(*g*) Pass the test flame in one direction only across the sample at intervals of 2° F. until the flashpoint is determined. Make the pass across the material in about 1 second.

(*h*) Repeat the test using a fresh sample and starting at least 20° F. below the previously determined flashpoint.

(3) *Results.* The two determinations should be within ±5° F. of each other or the test must be repeated.

c. Additional Information. ASTM Designations D 92 and D 1310 may be consulted for more specific details of apparatus and procedure for each of these tests.

3–16. Penetration Test

As indicated in paragraph 3-4*b*, the grade of an asphalt cement is based upon the results of the standard penetration test. Penetration is defined as the distance that a standard needle vertically penetrates a sample of the material under standard conditions of time, temperature, and loading. Standard conditions are 5 seconds, 25° C. (77° F.) and 100 grams, respectively, and the units of penetration are hundredths of a centimeter. Other conditions of temperature, load and time that are used for special testing are:

Temperature	*Load, g*	*Time, sec*
32° F. (0° C.)	200	60
39.2° F. (4° C.)	200	60
115° F. (46.1° C.)	50	5

a. Apparatus. Equipment required for this test is as follows:

Bath, hot water
Copper beaker, or frying pan
Stainless steel box
Electric hotplate
Sieve pan, 8-inch diameter
Penetrometer, asphalt testing (fig. 3–9)
Thermometer (66° to 80° F.)
Stopwatch

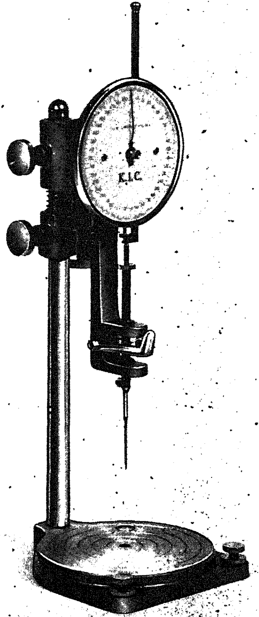


Figure 3–9. Asphalt penetrometer.

b. Procedure. The described test is dependent upon the use of the water bath maintained as closely as possible to the standard temperature of 77° F. Since the penetration of an asphalt cement varies with temperature, the bath is maintained at 77° F. In the event that this is impractical, or when less accuracy is required, the test may be performed with the sample at room temperature. The procedure of the test is as follows:

(1) Melt the sample at the lowest possible temperature, using the electric hotplate and frying pan or copper beaker. Stir thoroughly until sample is homogeneous and free of air bubbles.

(2) Pour the sample into the sample container (moisture box) to a depth of not less than $\frac{3}{8}$ inch for the harder grades, and $\frac{5}{8}$ inch for the softer grades. Protect the sample from dust and allow it to cool in in an atmosphere not lower than 65° F. for 1 hour.

(3) Place the sample in its container in the sieve pan (or any other small, flat-bottomed pan or container which is about 2 inches deep) and immerse the pan and sample in the water bath maintained at 77° F. (25° C.), for 1 hour.

(4) During the test, the sample remains in the pan which is filled with water from the water bath. The water should completely cover the sample container in order to maintain the temperature during the test. Place the transfer dish containing the sample in its container and the water on the stand of the penetrometer.

(5) Adjust the needle to make contact with the surface of the sample. Place a light in a position so that the actual needlepoint and its image reflected on the surface of the specimen are clearly defined. Contact may be judged when the point and its image touch on the surface.

(6) Either note the reading of the dial or bring the indicator on the dial to zero.

(7) The needle and plunger are designed to have a standard weight of 100 grams. Release the needle for a period of 5 seconds, after which, read the distance penetrated on the penetrometer. The dial is divided into 38 major divisions, each of which is in turn marked in 10 smaller subdivisions. Each of the smaller subdivisions represents a penetration of the needle of 1/100 centimeter. Thus, readings on th dial give penetration values without conversion.

(8) Test at least three points on the surfac not less than $\frac{3}{8}$ inch from the side of the containe and not less than $\frac{3}{8}$ inch apart. After each test, th sample and pan are returned to the water bath, an the needle is carefully wiped toward its point with clean, dry cloth to remove all adhering asphalt.

c. Results. The reported penetration is the averag of at least three tests whose values do not differ mor than four points between maximum and minimum

3–17. Solubility Test

This test indicates the purity of the asphalts. Th specimen is dissolved in carbon tetrachloride o carbon disulphide which should almost complete dissolve all the bitumen. The amount of residue i an indication of the amount of impurity in th specimen. Refer to ASTM Designation D 2042 fo complete instructions and precautions in making thi test.

3–18. Viscosity Test

a. Saybolt-Furol Method. This test measures th time in seconds required to pass 60 cc of liqui asphalt material at a given temperature through tube of standard dimensions. Its purpose is to determine the viscosity of the material from which th spraying temperatures are established for field application of the bitumen. The special apparatu needed and test procedures are given in ASTM Designation D 88.

b. Kinematic Viscosity Test. This test, like th Saybolt-Furol (*a* above), measures the time that given amount of liquid asphalt material will flo through a tube of standard dimensions under rigidl controlled conditions of temperature and pressur (or vacuum). The test establishes the viscosity of th liquid and when correlated with the specific gravit of the material at the same temperature, results i a numerical designation called kinematic viscosity The units used for kinematic viscosity are the *stoke* (square centimeters per second) or centistokes (10 stokes) (para 3–4). Determination of kinemati viscosity requires much special laboratory equipmen that is not available in the field. See ASTM Designation D 2170 and D 2171 for standard test method

Section V. TESTS ON AGGREGATE AND FILLERS

3–19. Discussion

The aggregate transmits the load, takes the abrasive wear of traffic, and provides a nonskid surface.

a. The desired characteristics include angular shape, rough surface, hardness and durability, resistance to water penetration, cleanliness, and gradation (para 3–12). Some of these characteristics, such as shape, surface, and cleanliness, are determine visually. Durability and hardness cannot be "seen" but require either knowledge based on experience o some form of abrasion testing (para 3–12). Rock that soak up water will eventually reach the conditio where the binder is forced from the surface pore and the cementing action breaks down (para 3–12) *b.* The remaining characteristics of gradation may be established to some extent by observation. However, the grain size distribution (sieve) tests will define the particle sizes and amounts much more accurately. In bituminous paving, the aggregate constitutes the bulk of the pavement. Common practice subdivides the bituminous aggregates into a coarse aggregate, a fine aggregate, and a mineral filler. The No. 10 sieve separates coarse from fine aggregate, and the No. 200 sieve size is the lower limit for fine aggregate. Usually 65 percent or more of mineral filler will pass the No. 200 sieve. The distribution of the different sizes determines the amount of voids that will remain, and in turn, aids in determining the amount of bitumen that will be needed.

c. Bituminous pavement specifications define acceptable gradation limits. The bitumen content for the mix is then determined from the trial mix properties which are defined in the specifications. The procedures for a bituminous mix design is described in paragraph 3–24. Detailed specifications for aggregates and filler for bituminous paving mixtures are given in ASTM Designations D 692, D 693, D 694, D 242, D 1073, and D 1139.

3–20. Sieve Analysis

A sieve analysis of the aggregates to be used in a paving mixture is required to determine the particle size distribution.

a. Amount Finer Than No. 200 Sieve. In bituminous paving, particles finer than the No. 200 sieve are referred to as mineral filler. To measure the amount of filler in a selected sample, a washed sieve analysis (para 2–26) is performed using the No. 40 and No. 200 sieves. After all the material possible has passed the No. 200 sieve and has been discarded, the materials on the sieves are returned to the original washed sample, oven-dried, and weighed. The amount of mineral filler is computed as:

$$\text{Percent finer than No. 200} = \frac{\text{original dry weight} - \text{washed dry weight}}{\text{original dry weight}} \times 100$$

b. Sieve Analysis of Fine and Coarse Aggregate (Washed). When definite amounts or limits of coarse and fine aggregate are specified, the sieve analysis with prewashing (para 2–26) must be made using suitable sieves. If no limits have been designated, a range of sieves is selected to give adequate information about the gradation. The results are recorded (DD Form 1206 (fig. 2–52)) and then plotted as a gradation curve (DD Form 1207 (fig. 2–54)). In testing aggregates, a representative sample is obtained by quartering (para 2–22), if necessary. The size of the sample depends somewhat on the maximum size of particles in the material.

Approximate aggregate size	*Approximate dry weight of sample, grams*
Fine aggregate	
At least 95% finer than No. 10*	100
At least 90% finer than No. 4 and more than 5% coarser than No. 10*	500
Coarse aggregate	
⅜-inch maximum size	1, 000
½-inch maximum size	2, 500
¾-inch maximum size	5, 000
1-inch maximum size	10, 000
1½-inch maximum size	15, 000
2-inch maximum size	20, 000
2½-inch maximum size	25, 000
3-inch maximum size	30, 000
3½-inch maximum size	35, 000

**Note.* ASTM specifies No. 8, 16, 30, and 50 instead of No. 10, 40 and 60. Tests based on ASTM standards identify the appropriate sieve sizes.

3–21. Specific Gravity

The specific gravities of aggregates used in bituminous paving mixtures are required in the computation of the percent of air voids and percent voids filled with bitumens. Apparent specific gravity used with aggregate blends showing water absorption of less than 2½ percent is based upon the apparent volume of the material, which does not include those pore spaces in the aggregate which are permeable to water. Bulk-impregnated specific gravity is used for aggregate blends with 2½ percent or greater water absorption. The methods for determining absorption of aggregates are given in paragraph 4–10.

a. Apparent Specific Gravity. The apparent specific gravity may be determined using the method described in paragraph 2–35, or it may be determined using the Dunagan apparatus furnished with the concrete test set (para 1–3).

(1) The apparatus using the concrete test set includes the Dunagan apparatus (fig. 3–10), sieves (2, 1½, ½, ⅜, and Nos. 4, 10, 40, 60, 80, 100, 200), evaporating dishes, electric oven, and pans.

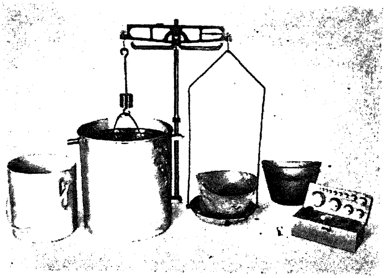


Figure 3–10. Specific gravity test, Dunagan apparatus.

(2) The step-by-step test procedure is as follows:

(*a*) Approximately 5,000 grams of aggregate is selected from the sample, not including particles smaller than the ⅜-inch sieve. A ⅜-inch sieve is provided.

(*b*) The aggregate is washed to remove any dust or other coating and dried to constant weight in the oven. The total weight of oven-dry aggregate is recorded on DD Form 1216 (fig. 3–6).

(*c*) The aggregate is immersed in water at 15° to 25° C., for a period of 24 hours.

(*d*) After soaking, the sample is placed in the copper bucket which is filled with water, and bucket and aggregate are then turned sharply back and forth to assist in removing any air.

(*e*) The bucket is suspended from the brass hanger and the water level brought up to the overflow pipe.

(*f*) The submerged weight is determined using weights placed in the scoop on the right-hand pan. Weights are recorded.

(3) The calculations required for the determination of the apparent specific gravity of coarse aggregate are shown on the DD Form 1216 and are self-explanatory.

b. Apparent Specific Gravity of Fine Aggregate (*Calibrateed Flask*).

(1) The equipment required to perform this test consists of:

Balance, 2000-gm
Evaporating dish
Battery filler
Volumetric flask, 500-ml
Electric oven
Pan
Thermometer (0° to 300° F., in 1° gradua
tions)
Absorbent paper or cloth

This equipment, with the exception of the oven, shown in figure 3–11.

(2) The test procedure for particles finer tha the ⅜-inch sieve is as follows (data are recorded o DD Form 1208 (fig. 2–57)):

(*a*) A 500-milliliter volumetric flask is cal brated (para 2–32).

(*b*) A representative sample weighing abou 500 grams is dried to constant weight in the electr oven.

(*c*) After cooling, the oven-dry weight of th sample is determined, and recorded on DD For 1208.

(*d*) The sample is transferred to the 50 milliliter flask, being careful not to lose any of th material.

(*e*) Clean water is added until the level ju reaches the neck of the flask. The sample is then a lowed to soak for 24 hours.

(*f*) After the sample has been soaked, th flask is held by the neck and rolled back and fort on a smooth surface until air bubbles cease to com from the sample. This procedure is illustrated i figure 3–12.

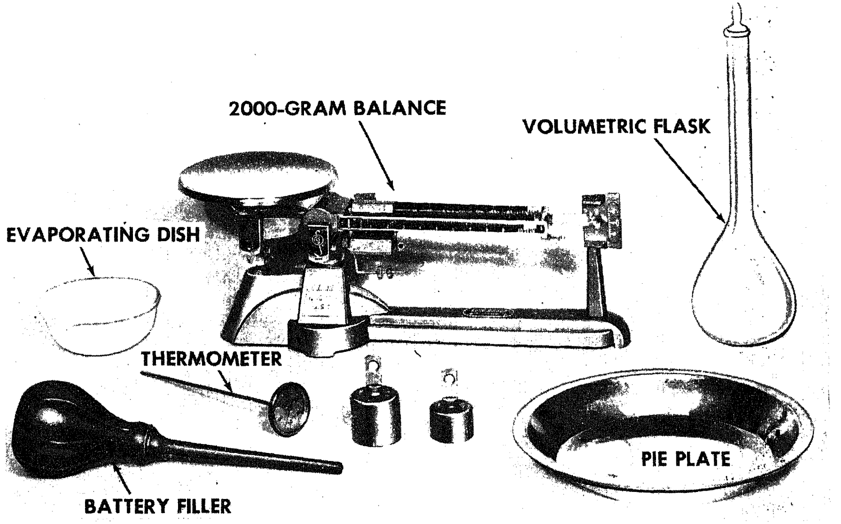


Figure 3–11. Calibrated flask apparatus for determining apparent specific gravity of fine aggregate.

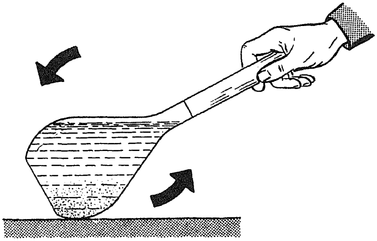


Figure 3–12. Manipulation of calibrated flask to remove air.

(*g*) When air has ceased to come from the sample, the battery filler is used to bring the water level up in the neck of the flask until the bottom of the meniscus coincides with the calibration mark on the flask. Absorbent paper or cloth is used to remove any drops of water from the inside of the neck, and on the outside of the flask.

(*h*) The weight of the flask, aggregate, and water is determined and recorded on DD Form 1208.

(*i*) The temperature is measured and recorded.

(3) The calculations for apparent specific gravity are the same as those indicated for soil in figure 2–57.

c. Apparent Specific Gravity of Fine Aggregate (*Uncalibrated Flask*). When a calibrated flask is not available, the following procedure can be used:

(1) The same equipment is required as for the test specified in *b* above, with the addition of a water bath maintained at 68° F. (20° C.). This procedure cannot be used if the temperature cannot be maintained.

(2) The detailed test procedure is as follows:

(*a*) A representative sample of aggregate passing the ⅜ inch sieve weighing about 500 grams is obtained and dried to constant weight in the electric oven maintained at at temperature of 212° F. (100° C.) to 230° F. (110° C.).

(*b*)After cooling in air, the dry weight is obtained and recorded.

(*c*) The sample is transferred to the flask, being careful not to lose any of the material.

(*d*) Clean water is added until its level just reaches the neck of the flask, and the sample is allowed to soak for 24 hours.

(*e*) After soaking for 24 hours, the flask is rolled back and forth on a smooth surface until air bubbles cease to come from the sample, as illustrated in figure 3–12.

(*f*) The battery filler is used to bring the water level up in the neck of the flask to slightly above the calibrated mark.

(*g*) The flask with water and aggregate is placed in the water bath maintained at a temperature of 68° F. (20° C.). The glass thermometer is used to check the temperature of the water in the flask from time to time.

(*h*) When the water in the flask reaches a uniform temperature of 68° F. (20° C.), the battery filler and absorbent paper are used to adjust the bottom of the meniscus to coincide with the calibration mark. Any drops of water inside of the neck of the flask are removed.

(*i*) The flask is removed from the bath and the outside thoroughly dried. The weight of the flask plus aggregate, plus the water at 68° F. (20° C.), is determined and recorded. It does not matter if the level of the water in the neck of the flask changes after removal from the bath. The proper adjustment was made at 68° F. (20° C.), and the total weight is not affected by the subsequent change in volume.

(*j*) The weight of the flask filled only with water at 68° F. (20° C.) must be known. Procedures in (*f*) through (*i*) above are repeated, using water only. The weight of flask filled with water at 68° F. (20° C.) is entered on DD Form 1216. This needs to be done only once for a given flask; this value can be tabulated and used in subsequent tests.

(3) Calculations necessary to the determination of the apparent specific gravity of fine aggregate using an uncalibrated flask are indicated on DD Form 1216 (fig. 3–6).

d. Bulk-Impregnated Specific Gravity of Aggregate. This method of test is intended for use in determining the specific gravity of the blended aggregates (including filler) used in hot asphaltic mixtures. This method is to be used only when the water absorption for the entire blend of aggregate selected for the job mix formula exceeds 2½ percent. The method is not applicable to determination of specific gravity of mineral filler except when included in the blended aggregate.

(1) The following equipment is required for performing this test:

Dunagan concrete test apparatus
Electric oven (sensitive to ±5° F. in the range of 275°–325° F.)
1-gallon capacity pails
Balance, 5-kg capacity sensitive to 0.1 g.
Bake pans, and
A heavy sheet metal strip for stirring contents of the pail.

A wire handle is convenient for handling the pail, but is not essential, since the container will be placed in the copper bucket of the Dunagan apparatus in order to determine weight submerged in water. A No. 10 can (an empty fruit or vegetable can) with the top smoothly cut out is satisfactory, but care must be taken to eliminate air entrapped under the bottom when the can is submerged.

(*a*) *Aggregate.* 1,000 grams of blended aggregate insuring that the sample represents prototype grading.

(*b*) *Asphalt.* 85–100 penetration grade asphalt cement.

(3) The procedure is as follows:

(*a*) Dry aggregate sample to constant weight at a temperature not less than 230° F. nor greater than 290° F. and after cooling in air, weigh to nearest 0.1 gram.

(b) Heat the asphalt to 280°±5° F. using care to insure that the temperature never exceeds 285° F. and add a sufficient amount to the 1-gallon pail to fill it about one-third full.

(*c*) Insert the sheet metal stirrer and allow pail and contents to cool to room temperature. (A minimum of 8 hours is usually required to accomplish this, and preferably overnight.)

(*d*) Weigh pails plus asphalt and stirrer in air at room temperature and submerged in water at 72° F.±2°.

(*e*) Place the pail of asphalt with stirrer and also the sample of aggregate in oven at 280°±5° F. until temperatures of both are equalized. (A minimum of 4 hours is usually required.)

(*f*) Remove aggregate and asphalt from oven, and gradually add aggregrate to asphalt, stirring thoroughly. After all aggregate is added, con- of mixing to end of stirring is 2 minutes. During th cooling period, apply a flame to the surface to remov air bubbles, if present. Cool to room temperatur (preferably overnight).

(*g*) Weigh in air and in water at 72°±2°

(4) Calculate the bulk impregnated specif gravity as follows:

$$\text{Bulk impregnated specific gravity} = \frac{A}{(D-E)-(B-C)}$$

Where:

A = weight of oven-dry aggregate in grams.
B = weight of pail plus stirrer plus asphalt in ai
C = weight of pail plus stirrer plus asphalt i water.
D = weight of pail plus stirrer plus asphalt plu aggregate in air.
E = weight of pail plus stirrer plus asphalt plu aggregate in water.

(5) Duplicate determinations should chec within 0.04. If the values are within the 0.04 tolerance, an average value should be used. If the initia duplicate tests are not within the 0.04 tolerance, repeat tests should be made in duplicate. If the secon set of test values is within the tolerance, the first tw test values should be discarded and an average valu of the second two tests should be used.

Section VI. BITUMINOUS MIX DESIGN

3–22. Introduction

Hot-mix bituminous concrete for pavements is a mixture of blended aggregate and filled with bituminous cement binder. The materials are heated when mixing to promote fluidity of the bitumen for thorough coverage of the aggregate particles. The design of a bituminous concrete mix consists of the determination of an economical blend and gradation of aggregates together with the necessary content of bituminous cement to produce a mixture that will be durable, have the stability to withstand traffic loads, and be workable for placement and compaction with the construction equipment available.

a. The procedures described in this section are performed during the design of a hot-mix bituminous concrete. They include testing, plotting the results on graphs, and checking the readings against values from the design tables. Testing of the ingredients and the mix is started before and continued throughout the paving operations. TM 5–330 more fully describes the design considerations. TM 5–337 explains the paving operations, and the blending and bitumen content criteria. This manual discusses the testing phases.

b. The following is a list of the general procedure in the design of a hot-mix. The objective is to determine the most economical blend of components whic will produce a final product that meets specifie requirements.

(1) Prepare a sieve analysis of each of the aggregates available.

(2) Determine the aggregate blend which wil achieve the specified gradation (TM 5–337). Plo the selected blend proportions on a graph with th allowable limits to see that it conforms.

(3) Determine the specific gravity of th components.

(4) Using selected percentages of bitumen (TM 5–337) make trial mixes and compute the design tes properties of the mix.

(5) Plot the test properties on individual graph using the selected bitumen percentages. Draw smooth curves through the plotted points.

(6) Select the optimum bitumen content fo each test property from the curves (as explained i the criteria tables).

(7) Average the bitumen content values (from (6) above), and from the graphs, read the tes property value corresponding to this average.

(8) Check these read values with the satisfactoriness-of-mix criteria.

c. The selection of the mix ratios of materials is tentative. The bitumen should be the same as the one to be used in the construction. The aggregates and fillers must meet definite requirements. In general, several blends should be considered for laboratory mix-design tests.

d. Gradation specifications are based on limits established by the Corps of Engineers as satisfactory. Within these limits, the following variables are considerations which will affect the final mix-design:

(1) Use of mix (surface course, binder course, or road mix).

(2) Binder (asphalt, cement, or tar).

(3) Loading (low tire pressure—100 psi and under, or high tire pressure—over 100 psi).

(4) Maximum size of aggregate (in stockpile, or based on thickness of the pavement course).

e. Once the gradation specifications have been selected, the available materials are checked to determine how to proportion the blend to meet these specifications. Sieve analyses of the available aggregates are studied and a series of trial blends is computed. Adjustment of the blend may have to be made after the design and prepared mix is tested. The considerations for establishing and adjusting the blend are explained in TM 5–337.

f. The determination of optimum bitumen content is based on a definite design and testing procedure, known as the Marshall method. This is outlined later in this section.

g. The final step is the preparation of a job-mix formula to be furnished to the construction unit.

h. It is recognized that at times it will be necessary to shorten the design procedure as much as possible to expedite military construction. Suggestions for expediting the design of the mix are given in paragraph 3–28. More detailed instructions are given in TM 5–337.

3–23. Apparatus—Marshall Test Method

The following equipment is required for mix design:

Balance, solution, 20-kg capacity—L
Balance, 2,000-gram capacity—P
Bath, hot water, electric—S
Beaker, copper—V
Bituminous test apparatus, Marshall Method—N consisting of:
  2 ea. collar, compaction cylinder, stability test
  6 ea. cylinder, compaction, stability test—G
  1 ea. head, test, Marshall stability—O
  1 ea. jack and frame, extrusion, stability test—Z
  1 ea. meter, flow stability—O
  1 ea. piston, extrusion, stability test—I
  4 ea. plate, base, compaction cylinder, stability test
  1 ea. tamper, compaction, stability test—F
  2 ea. adapter, ring, penetrometer—W
Compaction hammer
Cone test apparatus—C
Cover, sieve, 8-inch diameter
Dishes, evaporating—U
Frame, testing sieve, Tyler rocker type
Gloves, asbestos—M
Hotplate, electric—A
Mixer, electric, complete with mixing bowl and beaters—J
Molds, 4-inch diameter
Oven, electric
Pan, bake—D
Pan, sieve, 8-inch diameter
Paper, filter—AA
Pedestal, compaction
Scoop, kitchen—K
Shaker, sieve, electric—T
Sieve, testing, 8-inch diameter Nos. 4, 10, 20, 40, 60, 100, and 200—B (See *Note* in para 2–24)
Spatula, 6-inch blade—H
Spoon, cooking—Y
Thermometer, dial, general laboratory (50°—300° F. in 2° increments)—E
Thermometer, asphalt testing, copper-sheathed point—Q
Tongs, beaker—R
Watch, stop—X
Rags
Kerosene or other petroleum distillate

Items in the above list which are followed by letters are shown in figures 3–13 and 3–14.

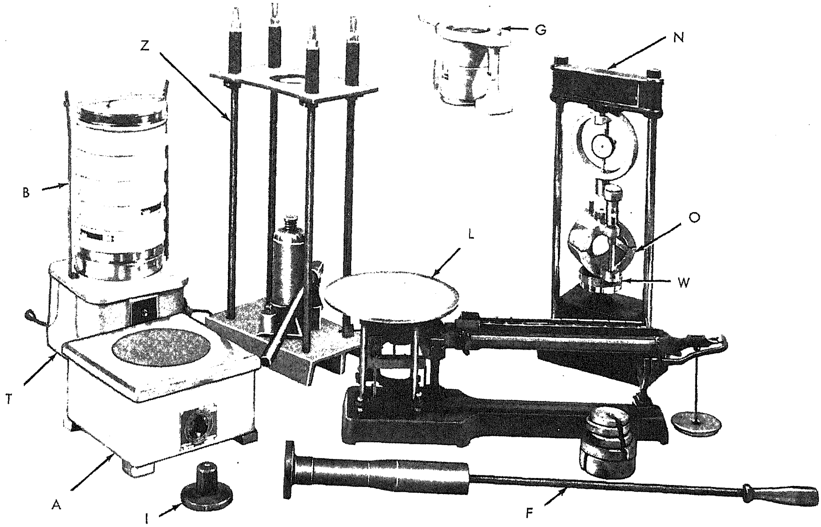


Figure 3-13. Apparatus for bituminous mix design—Marshall method.

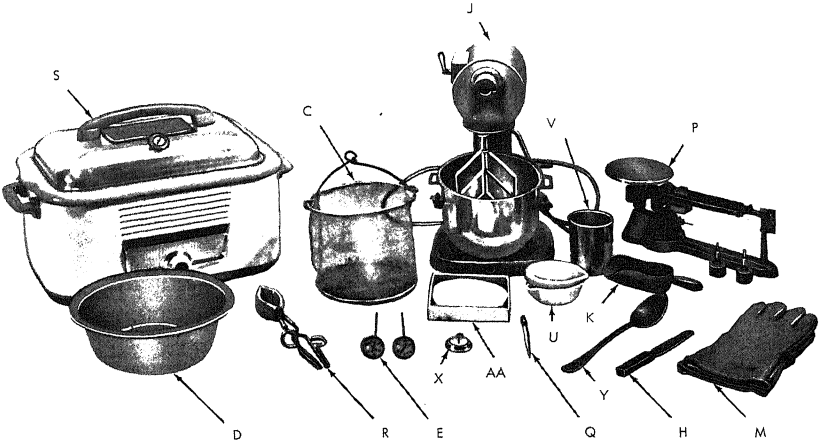


Figure 3-14. Apparatus for bituminous mix design—Marshall method.

-24. Procedures—Marshall Method

he Marshall method described is applicable to hot-ix mixtures using penetration grades of asphalt ement and containing aggregate with not more than) percent of the aggregate larger than the 1-inch eve. The following procedures and example provide or the determination of the optimum asphalt conent for one particular blend of aggregates.

a. Selection of Asphalt Contents. The optimum mount of asphalt cement for the aggregate to be ested must be estimated in order to start the labratory tests. Tests are continued until results show t least two asphalt contents above and two below hat would be the optimum content. Since the optiium is not determined until after the results are lotted, specimens are prepared usually for each of x different contents. At least four specimens will e prepared at each asphalt content. One percent icremental changes of asphalt content may be used or preliminary work; however, increments of ½ f 1 percent will be used for final tests. The percent f asphalt will be expressed as percent of the total eight of the batch of paving mix. TM 5-337 exlains how to establish the estimated optimum ontent.

b. Preparation of Aggregates. About 30 kg or 70 ounds of the selected blend are dried to constant eight at 110°±5° C. (221° to 239° F.). This mount of material is equivalent to 1200 grams per pecimen for four specimens per content and for six sphalt contents (total of 24 specimens). The dry ggregate is separated into several size ranges by ieving. Preparing test specimens by recombining eparate size fractions prevents nonuniformity of test pecimens which results from aggregate segregation uring handling and heating. Separations may vary, epending on the grading of the aggregate. Incomlete sieving of the different sizes is not detrimental f not carried out to excess, providing it is recogized and appropriate adjustments are made in ecombining the aggregate for preparing the test mixures. A sieve analysis of each of the separate fracions is made to determine the adequacy of the eparation and to furnish the necessary data for calulating the percentage of each size range to use in reparing the required mix.

c. Preparation of Mix. The bitumen and the agregates must be heated to specified temperatures or mixing. These temperatures are based on the itumen which will be used. Bitumen should not be held at the mixing temperature for more than 1 hour before using. Therefore, the preparation should be planned so that the mixing will be done within this time limit. The mixing pans are preheated to a temperature about 50 F. above the mixing temperature. The heated dry aggregate fractions are poured into the pans and mixed thoroughly. A crater is formed in the mixed aggregate and the required amount of bitumen (*a* above) at the proper temperature is poured into the crater. At this point, the temperature of the ingredients should be within the limits specified above. Mixing of the aggregates and bitumen should be as rapid and as thorough as possible to get a uniform distribution of the bitumen throughout the aggregate.

Bitumen type	*Mixing temp. °F.*	
	Aggregate	*Bitumen*
sphalt cement	300±5	270±5
'ar (RT 10, 11, or 12)	225±5	200±5
ubberized tar	250±5	225±5

d. Preparation and Compaction of Specimens. Four specimens are to be prepared at each bitumen content and the molds should be ready to receive the specimens as soon as they are mixed. The striking face of the compaction hammers and the compaction molds must be thoroughly clean and heated to 180° to 250° F. Oiling the mold and other metal in contact with the mix before the mixture is introduced facilitates removal of the specimen after compaction. The mix is placed in the mold (rodding the material as it is added), after which the collar is removed, and the top surface of the mix smoothed to a slightly rounded shape with a trowel. (The thickness of the compacted specimen should be 2½ ±⅛ inch. One or two trials will indicate the quantity of mix required to produce such a specimen.) The collar is replaced and the mold assembly placed on the compaction pedestal. The temperature at this point must be as specified for compaction.

Bitumen type	*Compaction temperature °F.*
Asphalt cement	250±5
Tar, RT-10, 11, or 12	180±5
Rubberized tar	200±5

The required number of blows are applied with the compaction hammer (fig. 3-15). The base plate and collar are removed and the mold reversed and reassembled. The required number of blows are then applied to the other side of the specimen. *For example*, roads, streets, and facilities for an airfield designed for aircraft whose tires carry 100 p.s.i. (or less) should be compacted by 50 blows on each end of the specimen. If the pavement is being designed for aircraft which carry tires with pressure greater than 100 p.s.i., the compactive effort should be 75 blows per side.

e. Cooling of Specimens. After compaction, the base plate and collar are removed, and the mold and the specimen are either air-cooled (normally overnight) or placed in cold water for a minimum of 2 minutes for fast cooling. The cooled specimen is

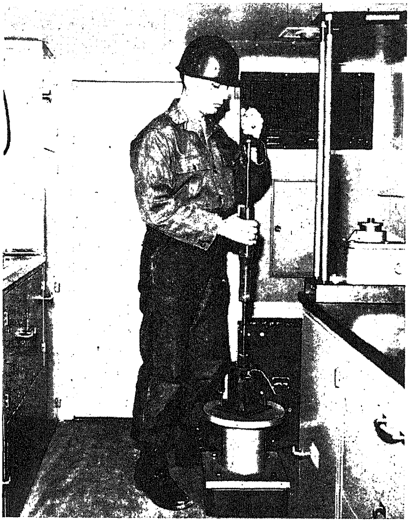


Figure 3–15. Compaction of bituminous trial mix specimens.

removed from the mold by an extension jack, or by placing the collar on the floor, the mold and specimen on top, and forcing the specimen out with light blows from the compaction hammer. The specimen, carefully handled and suitably identified, is placed on a smooth well-leveled surface until ready for testing. It is ready for testing anytime after it has cooled to room temperature.

f. Density and Voids Determination. Density of the specimens is determined by weighing in air and in water (fig. 3–16). A direct weight in water of open textured or porous specimens will give erroneou results because of absorption of water, and othe means must be used to determine the volume of th specimen. One means of measuring the volume of th porous specimens is to coat the specimen with paraffi to seal all the voids and then weigh the coated specimen in air and in water. (See fig. 2–27 for typical dat sheet.) A correction is made for the weight an volume of paraffin. The difference between these tw weights in grams gives the volume of the specime in cubic centimeters. The unit weight of the mix i pounds per cubic foot for each specimen will b determined by multiplying the specific gravity o the specimen by 62.4. Before the calculations fo percent voids can be carried out, it is necessary t know the specific gravity of the aggregate blend, an the asphalt content used.

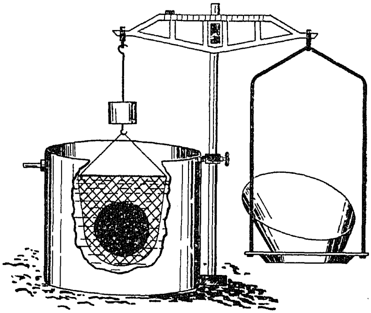


Figure 3–16. Weighing compacted bituminous specimen in water.

g. Stability and Flow Determination. The tes specimens are brought to the desired temperature o test by immersing in a water bath for at least 2 minutes, and not more than 1 hour. The bath temperature for asphalt samples is 140°±1°F.; for ta samples (RT–10 to RT–12), 100°±1°F.; for tar rubber samples, 120°±1° F.

(1) The inside surfaces of the test heads an the guide rods are cleaned thoroughly prior to making the stability test and the guide rods well lubricated so that the upper test head will slide easily ove the guide rods on the lower test head.

(2) The specimen is removed from the wate bath and placed on its side in the lower section of th breaking head. The upper section of the breakin head is positioned on the guide rods and on the specimen, and the complete assembly then placed in position in the testing machine (fig. 3–17). In order t prevent excessive cooling of this specimen with resulting increase in stability value, the entire procedure from the time the specimen is removed from th water bath should be performed as quickly as possible; normally in 30 seconds.

(3) The flowmeter is placed over one of th guide rods and an initial reading taken on the flowmeter is estimated to 0.01 inch. The flowmeter is hel firmly over the guide rod while loading the specime and is read or removed from its position over th guide rod just when the load first begins to decreas as indicated by the dial gage in the proving ring.

(4) Load is applied to the specimen at a consta rate of strain of 2 inches per minute until failure o the specimen occurs. The load builds up on the typic test as movement occurs, until it reaches a maximu and falls off. The maximum reading of the dial, converted to pounds, is the stability value for th specimen.

(5) A second reading is taken on the flowmete and the difference between this reading and th

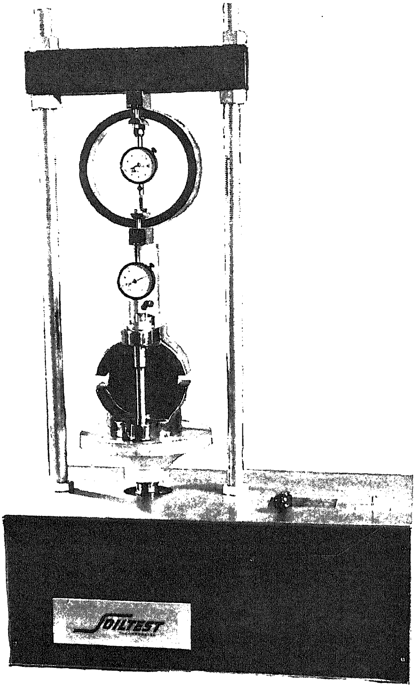


Figure 3-17. Stability test assembly.

original reading, expressed in hundredths of an inch, is the flow value of the specimen.

(6) The stability value varies directly with the thickness of the specimen. Therefore it is necessary to correct the stability values for specimens of a thickness greater or less than the standard 2½ inches. Table 3-5 shows the necessary conversion factors for specimens varying in thickness from 1 inch to 3 inches. Table 3-5 also contains data whereby the stability conversion factor can be determined on the basis of the volume of the specimen, since the volume is a direct function of height for a constant 4-inch diameter specimen. Flow values do not vary appreciably with change in thickness of specimen; therefore no corrections are needed for the flow value. An example of the design is given in the following paragraph.

Table 3-5. Stability Correlation Ratios, Marshall Stability Test

Volume of specimen in cubic centimeters	Approximate thickness of specimen in inches	Correlation ratio
200-213	1	5.56
214-225	1 1/16	5.00
226-237	1 1/8	4.55
238-250	1 3/16	4.17
251-264	1 1/4	3.85
265-276	1 5/16	3.57
277-289	1 3/8	3.33
290-301	1 7/16	3.03
302-316	1 1/2	2.78
317-328	1 9/16	2.50
329-340	1 5/8	2.27
341-353	1 11/16	2.08
354-367	1 3/4	1.92
368-379	1 13/16	1.79
380-392	1 7/8	1.67
393-405	1 15/16	1.56
406-420	2	1.47
421-431	2 1/16	1.39
432-443	2 1/8	1.32
444-456	2 3/16	1.25
457-470	2 1/4	1.19
471-482	2 5/16	1.14
483-495	2 3/8	1.09
496-508	2 7/16	1.04
509-522	2 1/2	1.00
523-535	2 9/16	0.96
536-546	2 5/8	0.93
547-559	2 11/16	0.89
560-573	2 3/4	0.86
574-585	2 13/16	0.83
586-598	2 7/8	0.81
599-610	2 15/16	0.78
611-625	3	0.76

NOTES: 1. The measured stability of a specimen multiplied by the ratio for the thickness of the specimen equals the corrected stability for a 2½ in. specimen.

2. Volume-thickness relationship is based on a specimen diameter of 4 in.

3-25. Example of Marshall Method of Hot-Mix Design

A typical mix design is illustrated by the calculations and graphs shown on figures 3-18 through 3-23.

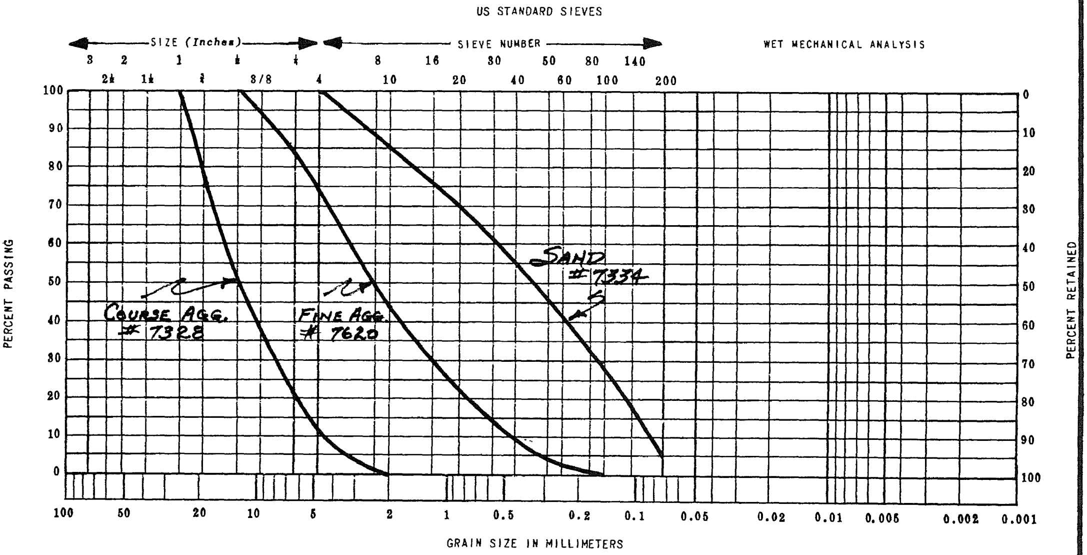
a. Aggregate Grading. An aggregate grading chart is shown in figure 3-18 (DD Form 1207). This chart shows the gradation curves of the three aggregates which are available for the mix. The sieve analyses are run in the manner described in paragraphs 3-19 and 3-20. Calculations may be made and data recorded on DD Form 1206 (fig. 2-52), preparatory to drawing curves. A gradation curve is not shown for the mineral filler to be used.

b. Aggregate Blending. A data and computation sheet for aggregate blending is shown in figure 3-19

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 2 APR 1970

PROJECT HIGHWAY #203



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS
	7328						COURSE AGGREGATE
	7620						FINE AGGREGATE
	7334						SAND

TECHNICIAN (Signature) | PLOTTED BY (Signature) | CHECKED BY (Signature)

(DD Form 1217). In the upper portion of the form, the gradation of the available aggregates is recorded. The lower portion may be used for the computation of the trial blend. Several attempts may be required before a blend meeting specifications is obtained. The cold feeds (quantities per batch) of aggregate to the asphalt plant are set according to the proportions obtained in computing the final trial blend.

c. Aggregate Blending Limits. Figure 3–20 is an aggregate grading chart (DD Form 1207) showing the specification limits for the mix and the gradation of the blend when mixed in the proportions shown in figure 3–19, Trial 1.

d. Specific Gravity of Bituminous Mix Components. A specific gravity worksheet is shown in figure 3–21 (DD Form 1216). This form is used for computing the specific gravity of all the bituminous mix components. If more aggregate fractions are used than are provided for on the form, additional forms may be used. Procedures for performing these tests are discussed in paragraph 3–21.

e. Marshall Stability Computations. DD Form 1218 (fig. 3–22) is a data and computation sheet used in the Marshall Stability Test. The specific gravity values of the aggregates (*d* above) and the aggregate fraction percentages from the trial blending (*b* above) furnish the information to compute the fractional weights and to prepare the test specimens. The measurements made on the test specimens (para 3–24) are recorded in the upper right-hand corner of the form. The stability, flow, unit weight of total mix, and percent voids filled with binder are determined to complete the form.

f. Marshall Method Computations. The values from DD Form 1218 (*e* above) for each binder-content computation are transferred to DD Form 1219 (fig. 3–23). Each graph on the form represents a different test property. The values for each property are plotted on their respective graph using the binder contents as ordinates. A smooth curve is drawn through the plotted points.

g. Marshall Test Criteria. Table 3–6 lists the criteria for determining optimum asphalt content (OAC). For each test property, the type of mix to be used and the expected load are considered. The optimum bitumen content for each property is designated as a definite point on the curve for that property. The bitumen content percentages (one for each property) are averaged and the average is used to read the corresponding value of each test property. The value, as determined, is referred to the criteria portion of table 3–6 to check if it is within the permissible limits so that the mix will perform satisfactorily.

h. Test Variation for Aggregates With 10 Percent or More Larger Than 1-Inch Maximum Size. The procedure described in paragraph 3–24 and the examples as given in *a* through *g* above are applicable to hot-mix design where the amount of aggregate larger than the 1 inch sieve is less than 10 percent of the total. When the larger than (plus) 1 inch material exceeds 10 percent of the total, the following variations are made in the procedure:

(1) Mix bitumen at the selected content with the *entire* aggregate, including the plus 1 inch portion,

(2) Pass the mixed hot batch through a 1 inch sieve. Discard the plus 1 inch portion.

(3) Make compacted specimens from the portion which passes the 1-inch sieve and perform the Marshall test, except do not calculate the voids of the compacted specimens at this time.

(4) Determine the bulk specific gravity of the plus 1 inch aggregate, and with the specific gravity of the compacted specimens, compute the adjusted specific gravity (G_A) as follows

$$G_A = \frac{100}{\frac{A}{C} + \frac{B}{D}} \times f$$

Where:

A = weight of dry, plus 1 inch material expressed as a percentage of the total batch weight (bitumen plus aggregate)

B = portion of total batch remaining after the dry, plus 1 inch portion is removed (100%—A %)

C = bulk specific gravity of plus 1 inch aggregate

D = actual specific gravity of compacted specimen

f = empirical factor = 0.995

(5) Calculate the voids by using the adjusted specific gravity and apply the design criteria for this value.

(6) Use stability and flow values as measured on the compacted specimens.

BITUMINOUS MIX DESIGN - AGGREGATE BLENDING		DATE 2 APR 1970
PROJECT HIGHWAY #203	JOB NO. 47236	AGGREGATE GRADATION NUMBER 2A

GRADATION OF MATERIAL

SIEVE SIZE (To be entered by Technician): →	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	
MATERIAL USED	PERCENT PASSING											
COURSE AGGREGATE (CA)	100	72	46	33	12	2	0	0	0	0	0	
FINE AGGREGATE (FA)	100	100	98	94	75	54	33	13	2	0	0	
FINE RIVER BAR SAND (FRBS)	100	100	100	100	100	98	90	76	58	35	3	
LIMESTONE DUST (LSD)	100	100	100	100	100	100	100	100	98	95	90	
DESIRED:	100	80-95	68-86	60-77	45-60	34-49	26-40	19-30	14-23	8-16	3-7	

COMBINED GRADATION FOR BLEND - TRIAL NUMBER FINAL

SIEVE SIZE (To be entered by Technician): →		1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	
MATERIAL USED	% USED	PERCENT PASSING											
CA	45	45.0	32.4	20.7	14.9	5.4	0.9	0	0	0	0	0	
FA	30	30.0	30.0	29.4	28.2	22.5	16.2	9.9	3.9	0.6	0	0	
FRBS	20	20.0	20.0	20.0	20.0	20.0	19.6	18.0	15.2	11.6	7.0	0.6	
LSD	5	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	4.9	4.8	4.5	
BLEND:		100.0	87.4	75.1	68.1	52.9	41.7	32.9	24.1	17.1	11.8	5.1	
DESIRED:		100.0	87.5	77.0	68.5	52.5	41.5	33.0	24.5	18.5	12.0	5.0	

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ______

SIEVE SIZE (To be entered by Technician): →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

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Figure 3-10. Bituminous mix design, aggregate blending data sheet.

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE (*To be entered by Technician*): →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE (*To be entered by Technician*): →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

REMARKS

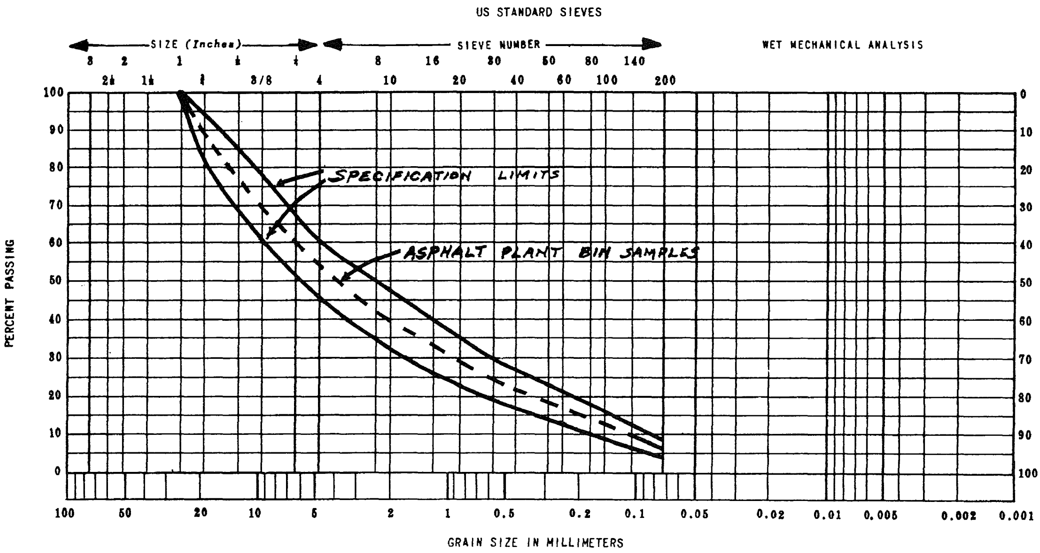
TECHNICIAN (*Signature*) Sp-4 W. D. Atkins	COMPUTED BY (*Signature*) Sgt. George M. Brown	CHECKED BY (*Signature*) 2/Lt. Donald M. Scott

Figure 3–19. Bituminous mix design, aggregate blending, data sheet—Continued

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADING CHART

DATE 2 APR 1970

PROJECT HIGHWAY #203



EXCAVATION NUMBER	SAMPLE NUMBER	NATURAL % MOISTURE	w_L	w_P	I_P	CLASSIFICATION	REMARKS

TECHNICIAN (Signature) SP-4 A. McMurray

PLOTTED BY (Signature) SP-4 A McMurray

CHECKED BY (Signature) Sgt. Arthur Price

DD FORM 1 AUG 57 1207

GPO 943037

Figure 3-20. Aggregate grading chart, specification limits and gradation of blended aggregate.

SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS	DATE 2 APR 1970			
PROJECT HIGHWAY #203	JOB No. 47236			
COARSE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING 1" SIEVE AND RETAINED ON 16 SIEVE				
SAMPLE NUMBER	CA			
1. WEIGHT OF OVEN - DRY AGGREGATE	378.3			
2. WEIGHT OF SATURATED AGGREGATE IN WATER	241.0			
3. DIFFERENCE *(Line 1 minus 2)*	137.3			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 1)}}{\text{(Line 3)}}$	$\frac{378.3}{137.3} = 2.755$			
FINE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING NUMBER 3/8" SIEVE				
SAMPLE NUMBER	FRBS			
4. WEIGHT OF OVEN - DRY MATERIAL	478.8			
5. WEIGHT OF FLASK FILLED WITH WATER AT 20°C	678.6			
6. SUM (Line 4 + 5)	1157.4			
7. WEIGHT OF FLASK + AGGREGATE + WATER AT 20°C,	977.4			
8. DIFFERENCE *(Line 6 minus 7)*	180.0			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 4)}}{\text{(Line 8)}}$	$\frac{478.8}{180.0} = 2.660$			
FILLER	**UNITS** *(Grams)*			
SAMPLE NUMBER	LSD			
9. WEIGHT OF OVEN - DRY MATERIAL	466.5			
10. WEIGHT OF FLASK FILLED WITH WATER AT 20°C,	676.1			
11. SUM *(Line 9 + 10)*	1142.6			
12. WEIGHT OF FLASK + AGGREGATE + WATER AT 20°C,	973.8			
13. DIFFERENCE *(Line 11 minus 12)*	168.8			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 9)}}{\text{(Line 13)}}$	$\frac{466.5}{168.8} = 2.762$			
BINDER	**UNITS** *(Grams)*			
SAMPLE NUMBER	6873			
14. WEIGHT OF PYCNOMETER FILLED WITH WATER	61.9595			
15. WEIGHT OF EMPTY PYCNOMETER	37.9215			
16. WEIGHT OF WATER *(Line 14 minus 15)*	24.0380			
17. WEIGHT OF PYCNOMETER + BINDER	47.8617			
18. WEIGHT OF BINDER *(Line 17 minus 15)*	9.9402			
19. WEIGHT OF PYCNOMETER + BINDER + WATER TO FILL PYCNOMETER	62.1568			
20. WEIGHT OF WATER TO FILL PYCNOMETER *(Line 19 minus 17)*	14.2951			
21. WEIGHT OF WATER DISPLACED BY BINDER *(Line 16 minus 20)*	9.7429			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 18)}}{\text{(Line 21)}}$	$\frac{9.9402}{9.7429} = 1.020$			

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*
Sp-4 A. McMurray		Sgt. Arthur Price

DD FORM 1 DEC 65 **1216** PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

Figure 3-21. Specific gravity of bituminous mix components, data sheet.

MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION 2 APR 1970

JOB NUMBER 47236

PROJECT HIGHWAY #203

DESCRIPTION OF BLEND SURFACE COURSE 45/30/20/5 AGG BLEND

SPECIMEN NUMBER	ASPHALT CEMENT *(Percent)*	THICKNESS *(Inches)*	WEIGHT *(Grams)*		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME *(Percent)*	VOIDS *(Percent)*		UNIT WEIGHT TOTAL MIX *(Lb./Cu.Ft.)*	STABILITY *(Pounds)*		FLOW UNITS OF 1/100 IN.
			IN AIR	IN WATER		ACTUAL	THEORIZED		TOTAL MIX	FILLED		MEASURED	CONVERTED	
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
					(d - e)	$\frac{(d)}{(f)}$		$\frac{(b \times g)}{(\text{Sp.Gr. of AC})}$	$(100-100\frac{(g)}{(h)})$	$\frac{(i)}{(i+j)}$	(g x 62.4)		*	
A-1	3.5		1228.3	716.3	512.0	2.339						2020	2020	11
A-2	3.5		1219.5	712.2	507.3	2.404						1862	1936	10
A-3	3.5		1205.5	705.3	500.2	2.410						1821	1894	8
A-4	3.5		1206.2	708.4	497.8	2.423						1892	1868	8
AVG.	3.5		—	—	—	2.409	2.579	8.3	6.6	55.7	150.3	—	1955	9
B-1	4.0		1276.9	747.3	529.6	2.411						2110	2026	10
B-2	4.0		1252.6	733.3	519.3	2.412						2025	2025	9
B-3	4.0		1243.5	730.7	512.8	2.425						1995	1995	9
B-4	4.0		1230.4	722.8	507.6	2.424						2080	2101	9
AVG.	4.0		—	—	—	2.418	2.550	9.5	5.5	66.3	150.9	—	2037	9
C-1	4.5		1254.4	738.2	516.2	2.430						2050	2050	12
C-2	4.5		1238.3	726.8	511.5	2.421						2095	2095	9
C-3	4.5		1239.0	724.9	514.1	2.410						2110	2110	10
C-4	4.5		1273.5	752.0	521.5	2.442						2045	2045	10
AVG.	4.5		—	—	—	2.426	2.539	10.7	4.5	70.4	151.4	—	2075	10

* *From conversion table*

COMPUTED BY Sp-4 Alejandria McMurray

CHECKED BY Sgt Arthur Price

DD FORM 1 DEC 65 1218 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

Figure 3-22. Test results, Marshall stability test.

MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

3 Apr. 1970

JOB NUMBER	PROJECT	DESCRIPTION OF BLEND

SPECIMEN NUMBER	ASPHALT CEMENT (Percent)	THICKNESS (Inches)	WEIGHT (Grams)		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME (Percent)	VOIDS (Percent)		UNIT WEIGHT TOTAL MIX (Lb./Cu.Ft.)	STABILITY (Pounds)		FLOW UNITS OF 1/100 IN
			IN AIR	IN WATER		ACTUAL	THEORIZED		TOTAL MIX	FILLED		MEASURED	CONVERTED	
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
					(d - e)	$\frac{(d)}{(f)}$		$\frac{(b \times g)}{(Sp.Gr.\ of\ AC)}$	$(100-100\frac{(g)}{(h)})$	$\frac{(i)}{(i+j)}$	$(g \times 62.4)$		*	
D-1	5.0		1237.9	727.0	510.9	2.423						1875	1875	14
D-2	5.0		1300.0	763.6	536.3	2.424						2130	1981	10
D-3	5.0		1273.6	746.9	526.7	2.418						1900	1824	12
D-4	5.0		1247.9	731.8	516.1	2.418						1855	1855	12
AVG.	5.0		—	—		2.421	2.519	11.9	3.9	75.3	151.5	—	1884	12
E-1	5.5		1237.3	724.1	513.2	2.411						1450	1450	12
E-2	5.5		1264.0	740.6	523.4	2.415						1530	1469	14
E-3	5.5		1286.4	752.4	534.0	2.409						1615	1550	13
E-4	5.5		1253.4	733.8	519.7	2.412						1505	1505	16
AVG.	5.5		—	—	—	2.412	2.500	13.0	3.6	78.3	150.5	—	1494	14

* From conversion table	COMPUTED BY Sp-4 Alexandria McMurray	CHECKED BY Sgt. Arthur Price

DD FORM 1 DEC 65 1218

PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

Figure 3-22. Test results, Marshall stability test—Continued.

BITUMINOUS MIX CURVES

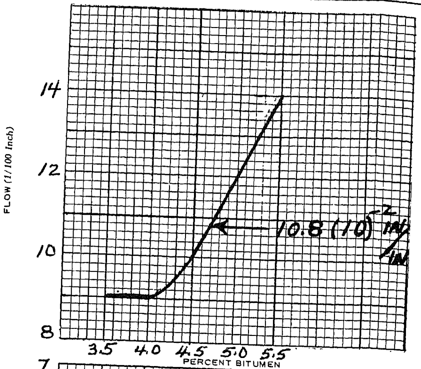
PROJECT AIRFIELD NO. 3

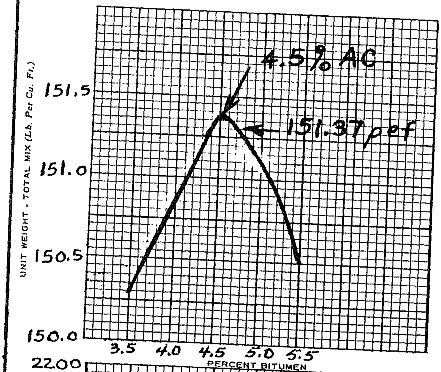
DATE 2 APR 1970

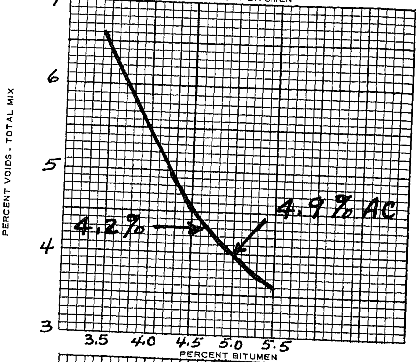
JOB NO. 47239 (SURFACE COURSE)

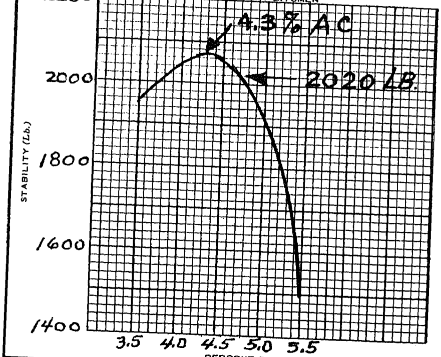
RECAP OF MARSHALL TEST RESULTS

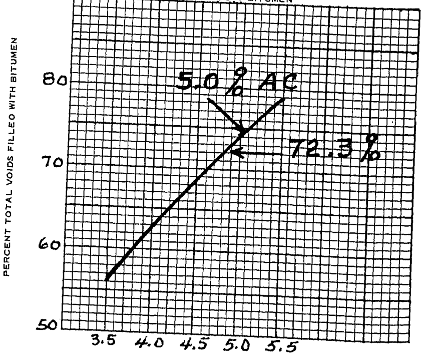
PERCENT BITUMEN CONTENT	3.5	4.0	4.5	5.0	5.5
UNIT WEIGHT - AGGREGATE ONLY					
UNIT WEIGHT - TOTAL MIX	150.3	150.9	151.4	151.1	150.5
STABILITY	1955	2037	2075	1884	1494
FLOW	9	9	10	12	14
PERCENT VOIDS - AGGREGATE ONLY					
PERCENT VOIDS - TOTAL MIX	6.6	5.5	4.5	3.9	3.6
PERCENT OF TOTAL VOIDS FILLED WITH BITUMEN	55.7	63.3	70.4	75.3	78.3











TECHNICIAN (Signature) Sp-4 A. McMurray

PLOTTED BY (Signature) Sp-4 A. McMurray

CHECKED BY (Signature) Sgt. Arthur Price

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Figure 3-23. Asphalt mix curves, Marshall test properties.

Table 3-6. Marshall Test Specifications and Determination of Optimum Asphalt Content

(1) Property	(2) Course	(3) Criteria (75 Blows) *** High press	(3) Criteria (50 Blows) Low press	(4) Determination of OAC High press	(4) Determination of OAC Low press
a. Aggregate blends showing water absorption up to 2 1/2% (used with ASTM apparent specific gravity).					
Stability	Surface	1800 or higher	500 or higher	Peak of curve	Peak of curve
Unit wt	Surface	---	---	Peak of curve	Peak of curve
Flow	Surface	16 or less	20 or less	Not used	Not used
% Voids total mix	Surface	3% - 5%	3% - 5%	4% -	4%
% Voids filled w/AC	Surface	70% - 80%	75% - 85%	75%	80%
Stability	Binder	1800 or higher	500 or higher	Peak of curve*	Peak of curve*
Unit wt	Binder	---	---	Peak of curve*	Peak of curve*
Flow	Binder	16 or less	20 or less	Not used	Not used
% Voids total mix	Binder	5% - 7%	4% - 6%	6%	5%
% Voids filled w/AC	Binder	50% - 70%	65% - 75%	60%*	70%*
Stability	Sand asphalt	**	500 or higher	**	Peak of curve
Unit wt	Sand asphalt	---	---	**	Peak of curve
Flow	Sand asphalt	**	20 or less	Not used	Not used
% Voids total mix	Sand asphalt	**	5% - 7%	**	6%
% Voids filled w/AC	Sand asphalt	**	65% - 75%	**	70%
b. Aggregate blends showing water absorption greater than 2 1/2% (used with bulk impregnated specific gravity).					
Stability	Surface	1800 or higher	500 or higher	Peak of curve	Peak of curve
Unit wt	Surface	---	---	Peak of curve	Peak of curve
Flow	Surface	16 or less	20 or less	Not used	Not used
% Voids total mix	Surface	2% - 4%	2% - 4%	3%	3%
% Voids filled w/AC	Surface	75% - 85%	80% - 90%	80%	85%
Stability	Binder	1800 or higher	500 or higher	Peak of curve*	Peak of curve*
Unit wt	Binder	---	---	Peak of curve*	Peak of curve*
Flow	Binder	16 or less	20 or less	Not used	Not used
% Voids total mix	Binder	4% - 6%	3% - 5%	5%	4%
% Voids filled w/AC	Binder	55% - 75%	70% - 80%	65%	75%
Stability	Sand asphalt	**	500 or higher	**	Peak of curve
Unit wt	Sand asphalt	---	---	**	Peak of curve
Flow	Sand asphalt	**	20 or less	Not used	Not used
% Voids total mix	Sand asphalt	**	4% - 6%	**	5%
% Voids filled w/AC	Sand asphalt	**	70% - 80%	**	75%

*If the inclusion of bitumen contents at these points in the average causes the voids total mix to fall outside the li
then the optimum bitumen should be adjusted so that the voids total mix are within the limits.

**Criteria for sand asphalt to be used in designing pavement for high pressure tires have not been established.

***High pressure tires are those above 100 psi. Low pressure tires are those with 100 psi or under.

3–26. Job-Mix Formula (AC Mixes)

When the mix has proven itself to be satisfactory, the percentages by weight of the aggregate and the averaged optimum bitumen content are combined to establish the job-mix formula. Figure 3–19 lists the final percentages of the aggregate for a given job-mix. By plotting the test results, figure 3–22, on DD Form 1219, figure 3–23, and applying the Marshall Test criteria for determining optimum bitumen content, the determination is made that the mix requires *4.7* percent of asphalt cement. Accordingly, the aggregates must be *95.3* percent of the total mix. The selected blend contained *45* percent coarse aggregate (CA), *30* percent fine aggregate (FA), *20* percent fine river bar sand (FRBS), and *5* percent limestone dust (LSD) mineral filler. The job-mix formula is computed as follows:

$$\begin{array}{rr}
CA = \underline{95.3} \times \underline{45} = & \underline{42.9\%} \\
FA = \underline{95.3} \times \underline{30} = & \underline{28.6\%} \\
FRBS = \underline{95.3} \times \underline{20} = & \underline{19.0\%} \\
\text{Mineral filler} = \underline{95.3} \times \underline{\ 5} = & \underline{4.8\%} \\
\hline
 & \underline{95.3\%} \\
\text{Asphalt cement} = & \underline{4.7\%} \\
\text{Total} = & 100.0\%
\end{array}$$

3–27. Modified Test for Cold-Mix Pavements

This method is used as an aid in determining the asphalt content for cold-mix design of light duty pavement. It can be used where asphalt cutbacks will be the binder. The procedures follow those used for hot mix design (para 3–24 *a* to *g*) in general with the following modifications:

a. Aggregates. These are dried to a moisture content expected during construction (up to a maximum of 2 percent, by weight).

b. Asphalt. The selected bitumen is mixed with the aggregates, but at the temperature recommended for field application. The aggregates remain at room temperature.

c. Curing. Prior to compaction, the mixture is cured for at least 12 hours in an oven set at 140°± 5° F.

d. Compaction. After curing, the mixture is compacted at 140° F. using 50 blows of the hammer at each end of the specimen.

e. Cooling. After molding, the specimens are cooled to room temperature in the molds. Care must be taken to remove the specimens, undisturbed and undamaged, from the molds.

f. Testing. The specimens are heated in an oven to 100°±2° F. and tested in the Marshall machine. Heating will normally take about 2 hours.

g. Selection of the Design Amount of Asphalt. The asphalt contents at maximum density and maximum stability, after averaging, are used as the design amount.

3–28. Surface Area Method of Mix Design

When laboratory equipment, except for sieve analysis, is not available, the following formulas may be used in place of laboratory procedures to determine the necessary asphalt content.

a. For Asphalt Cement,

$$P = 0.02a + 0.07b + 0.15c + 0.20d$$

Where

P= Percent (expressed as a whole number) of asphalt material *by weight of dry aggregate*

a= Percent (expressed as a whole number) of mineral aggregate retained on No. 50 sieve

b= Percent (expressed as a whole number) of mineral aggregate passing No. 50 and retained on No. 100 sieve

c= Percent (expressed as a whole number) of mineral aggregate passing No. 100 and retained on No. 200 sieve

d= Percent (expressed as a whole number) of mineral aggregate passing the No. 200 sieve

Absorptive aggregates such as slag, limerock, vesicular lava and coral will require additional asphalt.

b. For Asphalt Emulsion. Formula No. 2 for determination of estimated percent when using asphalt emulsions is as follows:

$$P = 0.05A + 0.1B + 0.5C$$

Where

P= Percent (expressed as a whole number) by weight asphalt emulsion, based on *weight of graded mineral aggregate*

A= Percent (expressed as a whole number) of mineral aggregate retained on No. 8 sieve

B= Percent (expressed as a whole number) of mineral aggregate passing No. 8 sieve, retained on No. 200 sieve

C= Percent (expressed as a whole number) of mineral aggregate passing No. 200 sieve

3–29. Plant Operation

Figures 3–24 and 3–25 are schematic drawings of a batch plant and a continuous mix plant, respectively.

a. It is generally necessary to combine aggregates from two or more sources to produce an aggregate mixture having the desired gradation. In the operation of a high-type bituminous paving plant, aggregates from the different sources are fed into the aggregate dryer in the approximate proportions required to produce the desired gradation. This initial proportioning generally is accomplished by means of a hopper-type feeder, operating from one or more bins, which feeds the aggregates into a cold elevator which in turn delivers them to the dryer. The mechanical feeder is loaded by a clamshell or other suitable means. The aggregates pass through the dryer where the moisture is driven off and the aggregates are heated to the desired temperatures. Upon leaving the dryer, the aggregate pass over vibrating screens where they are separated according to size. The usual screening equipment for a three-bin plant consists of a rejection (scalping) screen for eliminating oversized material and screens for dividing the coarse aggregate into two separate sizes (bins). Fine aggregate goes into the third bin. An additional screen is provided for further separation of the coarse aggregates in a four-bin plant. Additional mineral filler, if required, usually is stored, and weighed or proportioned into the mix separately. It may be obtained from the plant's dust collector or from an external source. Plant screens may be changed to provide a variation in size of openings; the sizes employed are largely dependent upon the type of mixture being produced. In some cases it may, be necessary to change the size of screens to obtain a proper balance of aggregate sizes in each bin.

b. The aggregates must be fed through the plant uniformly, preferably by mechanical feeder, in order to obtain efficient plant operation and produce a desired mixture. It is usually necessary to make some slight adjustments in the plant-bin proportions, since a screen analysis of the hot storage bins will not entirely duplicate the screen analysis used in the laboratory design. This may be a result of:

(1) Fines lost while passing through the dryer (unless the equipment includes an effective dust collector and the fines are returned to the mix).

(2) Aggregate degradation in the dryer.

(3) Plant screens which are not completely efficient in separation of the aggregate, with the result that some fines are carried over into the coarser bins.

(4) Separation of material at hot bins into more or fewer fractions than represented at stockpiles.

c. Intermediate-type plants will not have the refinements of the high-type plant described in *a* above. Such items as the gradation control unit, the dryer unit, and even the storage hoppers or bins may not be components of this type of plant. The aggregates from stockpiles or trucks are added directly (in correct proportion) into the elevator and then fed into the mixer. Corrections or changes in aggregate proportions must be made during the feed. If a dryer is not part of the plant, adjustments to compensate for moisture must be anticipated and made externally.

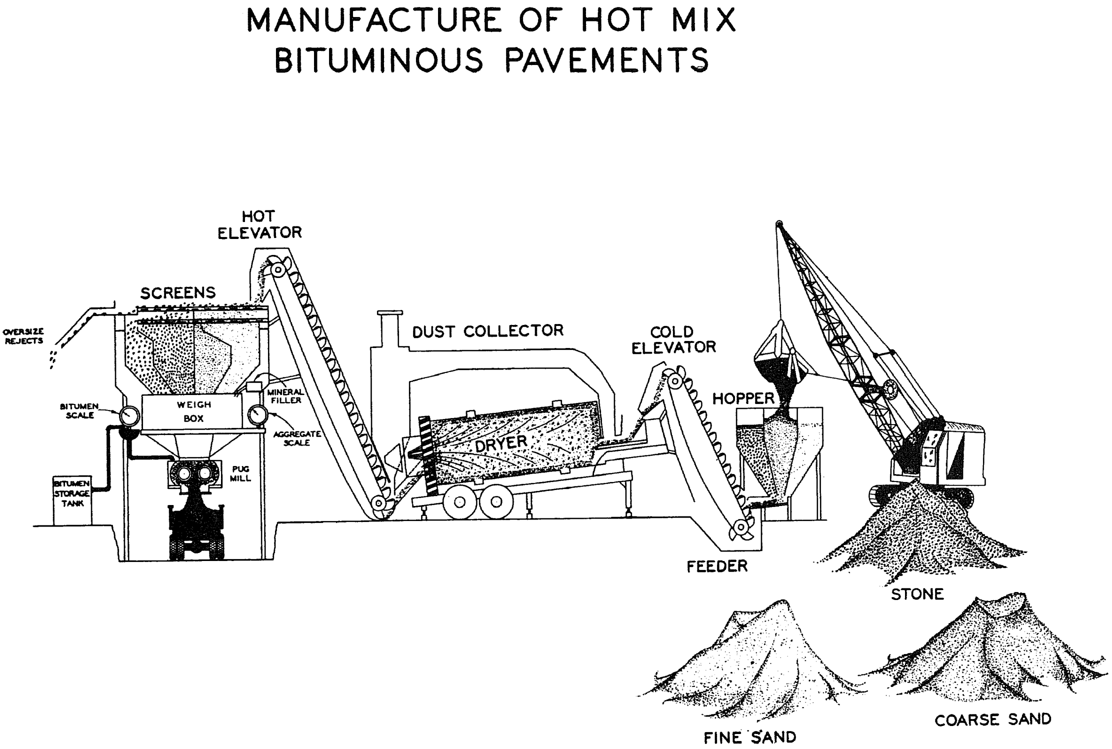

Figure 3-24. Bituminous hot-mix batch plant.

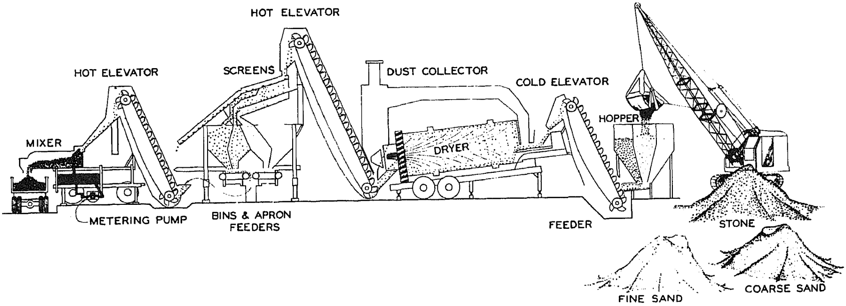

Figure 3–25. Bituminous hot-mix continuous-mix plant.

–30. Initiating Plant Production

The heaviest demands on laboratory facilities arise t the initiation of plant production. Preliminary omputations, which provide the gradation on which he mixture design is based, may be made to determine the weight of material from each bin. However, t should be recognized that the gradation of the ggregate supplied by the plant in accordance with omputed bin weights may not reproduce precisely he desired gradation. The gradation of the plant-produced aggregates generally approximates the one sed in design, within reasonable tolerances, if nitial sampling for design purposes has been ac-omplished properly and if the plant is operated fficiently. Certain steps should be taken, however, o insure that satisfactory mixtures are reproduced rom the beginning and throughout the period of plant production. Procedures outlined in this section will insure satisfactory paving mixtures.

a. Sieve Analysis. Sieve analysis (para 3–20) is made on material from each plant bin. Samples for hese sieve analyses are obtained after a few tons of ggregate have been processed through the dryer nd screens, in order that the sample will be repre-entative. Final bin proportions may be determined n the basis of these sieve analyses.

b. Provision for Redesign of Mix. The aggregates rom the bins sometimes cannot be proportioned to eproduce satisfactorily the gradation of the aggregate used in the laboratory design. It is then necessary to redesign the mix, using plant-produced ggregates. Specimens are prepared and tested for the new design in the same manner as for the original design tests. Optimum asphalt content and probable satisfactoriness of the mix that will be produced by the plant are determined thereby. Occasions may arise in which the gradation of the plant-produced aggregate will differ from the laboratory design so that a part of the aggregates may be wasted. Consideration then should be given to redesigning the mix to use all of the available aggregate. Sufficient additional tests should be performed to establish optimum asphalt requirements and insure that the mix meets applicable criteria for satisfactoriness.

3–31. Controlling Plant Production

a. A sample of paving mix adequate for the preparation of four test specimens is obtained from each of the first four truckloads of paving mix upon initiation of plant production. The four specimens prepared from each of these samples are compacted and tested in accordance with standard procedures described previously. The tests are conducted as rapidly as possible, and plant production delayed until data from these tests are available. These data must conform to final design data at the same asphalt content, within reasonable tolerances, before plant production is resumed. If necessary, proper adjustments are made to secure a conforming mix. Such procedures will delay plant production generally less than 2 hours, and production of satisfactory mixes is thereby assured from the start. As soon as data from tests of the plant-produced mix are obtained, they should be compared with corresponding design data for further adjustment of the mix, if necessary.

b. The average flow and stability values of truck samples should be in substantial agreement with flow and stability values from the final design. Variations of not more than two points in flow and not more than 10 percent in stability are allowable. The average unit weight of total mix of specimens prepared from samples obtained from trucks should not vary from the average unit weight of specimens prepared from plant-produced mixtures by more than 1.5 pounds per cubic foot. Plant production should be delayed, if these property variations exceed those noted above, until the cause of these variations is determined. Computations for scale weights and size of gate openings are checked first. If no error is found in these computations, the plant scales or other proportioning devices are recalibrated. Variations of only a few tenths of one percent in asphalt contents may cause variations of two or three points in the flow value. Small variations in aggregate quantities generally are not particularly effective in changing test properties. Plant scales or proportioning devices found to be inaccurate should be adjusted, and four additional truckloads of plant mix prepared, sampled, and tested. The plant must not be placed in continuous operation until the variations in test properties are within allowable tolerances. Once the plant has been placed in continuous operation, test specimens are prepared from approximately each 400 tons of mix placed. All the Marshall properties must be determined from these samples. Tolerances cited above are allowable for the continuation of plant production. Also, the variation of voids in the total mix must not exceed 1 percent.

c. Generally, a material increase in flow value indicates that either the gradation of the mix has changed sufficiently to require a revision in the optimum asphalt content for the mix, or too much asphalt is being incorporated in the mix. Substantial changes in stability or void content also serve as an indication of these factors. As a general rule, however, the flow and stability values are obtainable quickly and are reasonably reliable indicators of the consistency of the plant-produced mix. Mix proportions should be adjusted whenever any of the test properties fall outside of the specified tolerances. In the case of batch plants, failure of the operator to weigh accurately the required proportions of material or faulty scales are common causes of paving-mixture deficiencies. The total weight of each load of mixture produced should not vary more than 2 percent from the total load of the batch weights dumped into the truck. Improper weighing or faulty scales may be detected readily and corrective measures taken by maintaining close check of load weights. Other probable causes of paving-mixture deficiencies for both batch and continuous mixing plants are shown in figure 3-26. These deficiencies are observed at the plant. Other imperfections and their causes that may be encountered in placing the mix in the pavement are given in figure 3-27.

	Probable Causes of Deficiencies in Hot Plant Mix Paving Mixtures																												
Item	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	
Types of Deficiencies That May Be Encountered in Producing Hot Plant Mix Paving Mixtures	Aggregate scales out of adjustment	Bridging of hot aggregate in bin	Bitumen scales out of adjustment	Lack of proper weighing by operator	Faulty pug mill dump gate	Bitumen temperature too high	Drum feed not uniform	Improper dryer operation	Too little bitumen	Too much bitumen	Sampling method not uniform	Excessive moisture content of aggregate	Bin overflow pipes not functioning	Leaky bins	Segregation of aggregates in bins	Mixing time not uniform	Mineral filler feed not uniform	Aggregate temperature too high	Temperature mechanism out of adjustment	Improper feed of aggregates to dryer	Overrated dryer capacity	Faulty screen operation	Overloaded pug mill	Improper set on pug mill	Bitumen and aggregate feed out of adjustment	Bitumen pump not functioning properly	Not sufficient hot aggregate in storage bins	Aggregate gates not properly set	
Bitumen content fails to check job-mix formula	X		X	X	X				X	X	X		X	X		X	X			X				X	X	X	X	X	Types of Hot Mix Trouble
Gradation fails to check job-mix formula	X	X		X	X						X		X	X	X		X			X		X		X	X		X	X	
Poorly mixed loads					X	X										X							X	X					
Fat, rich mixtures	X		X	X	X	X				X	X			X			X			X					X	X		X	
Lean or burned mixtures	X		X	X	X	X		X	X		X			X				X	X	X					X	X		X	
Mixture temperature fails to check job mix						X	X	X										X	X		X								
Smoking loads							X	X										X	X	X									
Steaming loads							X	X				X							X	X	X								
Overweight or underweight loads		X	X	X	X														X										
Lack of uniformity of mixtures in loads		X	X	X	X	X			X	X		X	X	X	X	X	X		X	X	X	X	X	X	X		X	X	

Items 6 to 23 incl. are applicable to all types plants. Items 1 to 5 incl. and items 24 to 28 incl. are applicable to batch plants and volumetric plants respectively.

Figure 3-26. Probable causes of paving mixture deficiencies detected at plant.

PROBABLE CAUSES OF IMPERFECTIONS IN FINISHED PAVEMENTS

EXCESSIVE PRIMECOAT	IMPROPER PROPORTIONING	UNSATISFACTORY BATCHES IN LOAD	POOR HANDWORK BEHIND SPREADER	EXCESSIVE SEGREGATION IN LAYING	INADEQUATE ROLLING	POOR SPREADER OPERATION	MIXTURE TOO HOT OR BURNED	MIXTURE TOO COLD	ROLLING MIXTURE WHEN TOO HOT	ROLLING MIXTURE WHEN TOO COLD	POORLY GRADED MIXTURE	UNSTABLE BASECOURSE	FAULTY ALLOWANCE FOR COMPACTION	ROLLER STANDING ON HOT PAVEMENT	MIXTURE TOO COARSE	LACK OF BITUMEN IN MIXTURE	EXCESS OF BITUMEN IN MIXTURE	INADEQUATE CROSS ROLLING	NOT CUT BACK TO UNIFORM THICKNESS	EXCESSIVE MOISTURE IN MIXTURE	TYPES OF PAVEMENT IMPERFECTIONS THAT MAY BE ENCOUNTERED IN LAYING HOT PLANT MIX PAVING MIXTURES
X		X															X				BLEEDING
							X									X				X	BROWN, DEAD APPEARANCE
	X	X		X							X				X						POOR SURFACE TEXTURE
	X	X	X	X	X	X		X	X	X		X		X				X			ROUGH UNEVEN SURFACE
			X					X					X						X		UNEVEN LATERAL JOINTS
			X			X		X		X			X						X		UNEVEN LONGITUDINAL JOINTS
					X					X								X			ROLLER MARKS
	X	X					X	X	X		X						X			X	PUSHING
			X			X		X	X					X							WAVES
	X	X		X				X		X	X				X	X					CRACKING
	X	X	X	X		X		X							X						HONEYCOMB
			X		X	X			X			X						X			DISTORTION
		X				X		X								X					TEARING OF SURFACE DURING LAYING
	X	X															X				RICH OR FAT SPOTS

3–32. Asphalt Content (Use of Dulin-Rotarex Extraction Device)

The procedure outlined in paragraph 3–31 for control of plant production, based upon the test properties secured by the use of the Marshall apparatus, may be supplemented by the use of the Dulin-Rotarex extraction device. The measured asphalt content is compared with the optimum asphalt content previously established; discrepancies may be corrected by plant adjustments, if necessary. After the asphalt has been extracted, the remaining aggregates may be subjected to a sieve analysis for comparison with the previously selected aggregate gradation, with corrections again being made in the operation of the plant, as necessary.

a. Apparatus. The following equipment is necessary for determining the asphalt content of a bituminous plant mixture by the use of the Dulin-Rotarex extraction device:

Balance, 2,000-gram capacity
Beaker, 600-ml capacity
Varnish brush
Graduate, 1,000-ml glass
Evaporating dishes
Dulin-Rotarex centrifugal extractor, hand-driven w/pad
Filter ring (paper-basket type)
Electric hotplate
Electric oven
Bake pan
Solvent (trichloroethane), 5-gal. can
Spatula
Cooking spoon
Crucible tongs

The Dulin-Rotarex extraction device and related equipment are shown in figure 3–28.

b. Test Procedure. This test may be performed upon a representative sample taken from an individual batch or upon a composite sample representing several batches.

(1) After the plant is in normal operation, a suitable composite sample is obtained by combining 5-pound samples of the hot-mix taken every 2 hours.

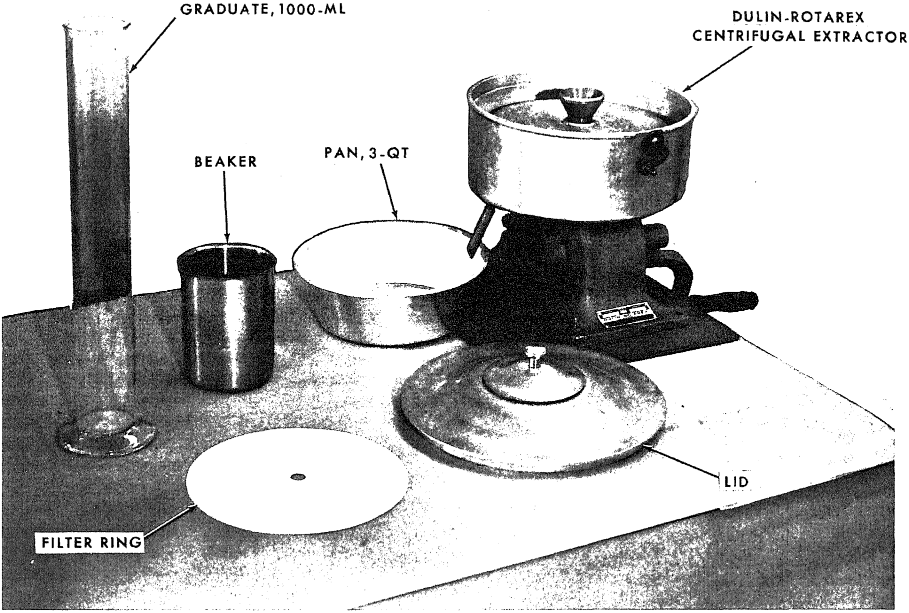

Figure 3-28. Dulin-Rotarex extraction equipment.

This is done by using a shovel and cutting completely across the stream of hot mixture as it is discharged from the mixer. The hot samples should be placed in a tin pail or other container with a tight-fitting lid until the last sample for the day is taken.

(2) The combined sample is heated in a pan on the hotplate or in the oven, until it is sufficiently soft to be easily disintegrated by the use of a large spoon. Care must be taken to see that the individual particles of aggregate are not crushed.

(3) A representative sample is prepared by the method of quartering. After cooling, not less than 500 grams of a sample containing particles larger than ¾ inch or not less than 200 grams of a sample with all particles smaller than ¾ inch are accurately weighed and placed in the bowl of the extraction device. The initial weight of the sample is recorded.

(4) A gasket-type filter paper is weighed and the weight recorder, after which it is fitted on the rim of the bowl. The cover plate is placed in position and securely locked in place. The bowl is then placed on the shaft of the extractor and fastened securely.

(5) The empty 1,000-milliliter glass graduate is placed under the spout. Approximately 150 cubic centimeters of solvent is poured into the bowl through the solvent funnel. Flow through the weep holes indicates a full bowl. The solvent is added from the beaker, pouring carefully. For best results, the solvent is allowed to set in the bowl for 10 to 15 minutes before operating.

(6) The lid is put on and the bowl rotated by turning the handcrank until the solvent discharges from the spout in a thin stream. After the first charge is drained out, the lid is removed and an additional amount of solvent is added. The lid is replaced and the bowl again rotated. This operation is repeated several times, until the discharged solvent is clean. With a little experience the operator can soon judge exactly what treatment is necessary for any given material.

(7) When the last addition of solvent has drained off, the bowl is removed and placed with the cover plate uppermost on a sheet of paper. The cover plate and the filter plate are carefully set aside on the piece of paper. After the sample is thoroughly

dry, it is carefully brushed from the bowl, cover plate, and filter paper, and weighed. This weight is recorded.

(8) Some of the mineral filler may be retained in the filter paper. For this reason, the filter paper is weighed, and the weight is recorded.

(9) The weight of the clean aggregate, plus the weight of any mineral filler retained in the filter paper, is subtracted from the weight of the original sample to determine the amount of asphalt extracted. This value for the amount of asphalt extracted is subject to correction, depending upon the amount of mineral filler contained in the solvent, as indicated below.

(10) The amount of mineral filler which passed through the filter paper is contained in the solvent and may be estimated as follows: The total amount of solvent, well stirred, is rapidly measured and a proportionate amount, usually from 50 to 100 cubic centimeters, poured into a previously weighed evaporating dish. The solvent is evaporated over *very low heat*, preferably outdoors. The residual material is then ignited and allowed to burn until all the organic material is consumed. The evaporating dish is allowed to cool and weighed again. If the organic material has been entirely burned off, the difference in initial and final weights of the dish represents the weight of mineral filler contained in that portion of the solvent used.

c. Calculations. The numerical example (fig. 3–29) will serve to illustrate the method of calculating the percentage of asphalt contained in the original sample. DD Form 1793 (Fig. 3–29 continued) will be reproduced locally for Army use on 8''x10½'' paper (image size $7\frac{4}{10}$''x$9\frac{5}{6}$''). Supply of DD Form 1793 for other services use will be requisitioned through normal publications supply channels.

d. Sieve Analysis of Aggregate. The clean aggregate may be subjected to sieve analysis, using the method described in paragraph 3–20. The amount of material passing the No. 200 sieve (mineral filler) in the sieve analysis must be increased by the weight of filler retained in the filter paper gasket contained in the solvent, determined as indicated in *c* above.

e. Testing Tar on Tar-Rubber Mixes. For tar or tar-rubber mixes the above procedure should be modified as follows:

(1) Soak sample in crystal-free creosote over night.

(2) Transfer soaked sample and creosote to Rotarex and centrifuge to remove the creosote.

(3) Wash sample with benzine until solvent is a light straw color.

DETERMINATION OF ASPHALT CONTENT
(Dulin - Rotarex Extractor)

PROJECT: HIGHWAY No. 206

DATE: 3 APR 1970

LINE NO.	ITEM		UNIT	1	2
1.	Weight of original sample and tare		g	253.1	
2.	Weight of tare		g	53.1	
3.	Weight of original sample	(1-2)	g	200.0	
4.	Weight of clean aggregate and tare		g	239.2	
5.	Weight of tare		g	53.1	
6.	Weight of clean aggregate	(4-5)	g	186.1	
7.	Final weight of filter paper		g	28.6	
8.	Initial weight of filter paper		g	27.2	
9.	Weight of filler in filter paper		g	1.4	
10.	Total amount of solvent		cm^3	927	
11.	Amount of solvent evaporated and ignited		cm^3	60	
12.	Final weight of evaporation dish and residue		g	33.3	
13.	Initial weight of clean evaporation dish		g	33.2	
14.	Amount of filler in evaporation dish	(12-13)	g	0.1	
15.	Amount of filler in total solvent	$(\frac{10}{11} \times 14)$	g	1.5	
16.	Total amount of aggregate in sample	(6+9+15)	g	189.0	
17.	Percentage asphalt	$(\frac{3-16}{6} \times 100)$	%	5.5	

REMARKS

SAMPLE 78

SPECIFICATION 5.5% CHECK, MIX O.K.

TECHNICIAN	COMPUTED BY	CHECKED BY
SP-4 Mickey	P. Simmons	R. Stephens

DD FORM 1 FEB 71 **1793**

Figure 3-29. Asphalt content by Dulin-Rotarex extractor, sample data sheet. (DD form 1793).

DETERMINATION OF ASPHALT CONTENT

(Dulin - Rotarex Extractor)

PROJECT

DATE

LINE NO.	ITEM		UNIT	1	2
1.	Weight of original sample and tare		g		
2.	Weight of tare		g		
3.	Weight of original sample	*(1 - 2)*	g		
4.	Weight of clean aggregate and tare		g		
5.	Weight of tare		g		
6.	Weight of clean aggregate	*(4 - 5)*	g		
7.	Final weight of filter paper		g		
8.	Initial weight of filter paper		g		
9.	Weight of filler in filter paper		g		
10.	Total amount of solvent		cm^3		
11.	Amount of solvent evaporated and ignited		cm^3		
12.	Final weight of evaporation dish and residue		g		
13.	Initial weight of clean evaporation dish		g		
14.	Amount of filler in evaporation dish	*(12 - 13)*	g		
15.	Amount of filler in total solvent	$(\frac{10}{11} \times 14)$	g		
16.	Total amount of aggregate in sample	*(6+9+15)*	g		
17.	Percentage asphalt	$(\frac{3-16}{6} \times 100)$	%		

REMARKS

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DD FORM 1 FEB 71 **1793**

Figure 3-29.—Continued.

3-33. Expediting the Design

When military expediency demands it, the preliminary laboratory mix designs are eliminated and the mix is designed directly from plant-produced aggregates. In such cases, the engineering officer will, on the basis of sieve analysis or other available information, or judgment, select the most promising of the available aggregates and start the plant using this aggregate. As pointed out previously, in nearly all cases the aggregate will show some breakdown of the particles because of the heating and screening; consequently the grading curve for the material which has passed through the plant nearly always will be somewhat different from grading curve of the material prior to entering the plant. Laboratory tests, using aggregate from the plant bins combined in the most desirable proportions must be conducted to determine the optimum asphalt content. Manufacture of the paving mix then can be initiated. If the available aggregate must be used, regardless of whether it produces a mix which meets all the design criteria at the optimum asphalt content, the asphalt content should be selected so as to give a mix which meets the criteria for flow and percent voids in the total mix, with the criteria for stability and percent voids filled with asphalt given less consideration.

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CONCRETE

CHAPTER 4

Section I CHARACTERISTICS AND BEHAVIOR

4–1. Description

a. Concrete is one of the most economical, versatile, and universally used construction materials. It is one of the few building materials that can be produced by the user directly on the job to meet his specific requirements. Concrete is an artificial stone which, when first mixed, forms a plastic or putty-like mixture. This mix is then placed into a form and allowed to harden or cure for a prescribed length of time. When cured, the finished concrete is a hard, stone-like material. It is used for pavements, foundations, dams and retaining walls, bridges, buildings of all types, and many military works.

b. Concrete can be described as a mixture of cement, fine and coarse aggregates, and water. During mixing, the cement and water from a fluid paste that permits thorough mixing and effective placement of the concrete. After mixing, the cement and water combine chemically to form a variety of compounds that adhere to the aggregate particles and bind the entire mass together into a strong, hard, stone-like material. This process, called hydration, causes a rapid development of strength in the first few hours after mixing, followed by less rapid gains in strength during the following weeks.

These "curing" conditions are essential to the development of the maximum strength of the concrete. The strength and desired resistance to weathering deterioration depend upon an adequate amount of cement-water paste to thoroughly cover all aggregate surfaces and fill the voids between aggregate particles. The amount of water per unit quantity of cement in the paste (water-cement ratio) largely determines the strength of the hardened concrete. For design purposes, the strength at 28 days after mixing usually is used. Beyond this period, the relative increase in strength becomes smaller and the strength curve flattens out. Some airfield designs use a 90-day strength which is determined by increasing the expected 28-day strength by 10 percent.

c. The aggregate is a mixture of fine and coarse material, generally sand with either natural gravel or crushed stone. It serves as an inert filler to provide the great bulk of material required. Well-graded aggregates contain particles of all sizes, from the largest permitted by the dimensions of the member to be formed, to sand fines. The smaller particles fill the spaces between the larger, thus providing a dense material that requires a minimum of cement paste for binder. The aggregate materials must be clean, hard, and at least as strong as the hardened cement to withstand the load pressures and the weathering processes.

4–2. Cement

Cement is a substance which hardens with time and holds or entraps objects or particles in a definite relation to each other. For concrete, portland cement usually is used. Portland cement is manufactured by a standardized process consisting of grinding limestone and clay, mixing them in correct proportions, heating the mixture to a high temperature to form clinkers, and pulverizing the clinkers so that 95 percent of the material will pass through a No. 200 sieve. A small amount of gypsum is added to retard the set. References to cement in Chapters 4 and 5 mean portland cement. The American Society for Testing and Materials specifies five common types of portland cement (ASTM Designation C 150). These are adequate for most purposes and can be used when necessary to compensate for the heat conditions encountered during the curing process and to offset soil conditions which can cause deterioration of the concrete. The various types of portland cement are known as hydraulic cements because they are capable of hardening and developing strength in the presence of water. The standard types of portland cement are—

a. Type I—Normal. This type is used for general construction when not subject to sulfate hazard and where hydration heat will not cause an objectionable rise in temperature.

b. Type II—Modified. This cement has a lower heat of hydration than the normal Type I and generates heat at a slower rate. It also has improved resistance to sulfate attack. Modified cement is used when concrete is placed in warm weather or in locations where a high temperature rise is objectionable. It is used in structures of considerable mass such as large piers, heavy abutments, and heavy retaining walls.

c. Type III—High-Early Strength. This type of cement is used where a high strength is needed very quickly. This may be due to a demand for early use, where forms must be removed as soon as possible, or in cold weather construction to reduce the period of protection against low or freezing temperatures.

d. Type IV—Low Heat. This cement is used where the amount and rate of heat generated must be kept to a minimum. Type IV cement develops strength at a slower rate than Type I cement but prevents the development of high temperatures in the structure with the attendant danger of shrinkage cracking upon later cooling. It is a special type, usually made to order, for massive structures such as large gravity dams wherein temperature rise from the heat of hydration is critical.

e. Type V—Sulfate Resistant. Sulfates react chemically with the cement compounds, disrupt the needed cementing action, and cause undesirable expansion of the mixture. The sulfates may be present in the water used to mix the concrete, or may be created by sulfurous gases from nearby industrial areas. The principal source of sulfate attack, however, occurs on foundations and other concrete in contact with the earth in certain regions and is caused by a reaction between the ground water, containing dissolved reactive minerals or acid, and the hardened cement. Resistant cement is low in calcium aluminate and is highly resistant to sulfate attack. It usually is available only on special order.

f. Other Types. These cements may be variations of the five types above or especially designed types.

(1) *Type IS—Portland blast furnace slag.* This cement uses granulated slag which is rapidly chilled or quenched from its molten state in water, steam, or air. The slag (from 25 to 65 percent of the total weight of cement) is ground with the cement clinker.

(2) *Type IP—Pozzolan.* This cement uses a mixture (15 to 35 or 40 percent) of pozzolan with the cement clinker. Pozzolan is a siliceous, or siliceous and aluminous material such as fly ash, volcanic ash, diatomaceous earth, or calcined shale. The strength of concrete made with pozzolanic cements is not as great as concrete made with the same amount of portland cement, but its workability may be better for some uses.

(3) *Air-entrained.* Concrete made with air-entrained cement is resistant to severe frost action and to salts used for ice and snow removal. It is produced by adding air-releasing materials to the clinker as it is ground. In general, air-entrainment may be controlled to a much greater extent by the use of admixtures with normal cements during mixing. This combination results in a concrete with tiny, distributed, and separated air bubbles (up to millions per cubic foot). The entrained air bubbles improve the workability of the fresh concrete. These bubbles reduce the capillary and water channel structure within the hardened concrete and restrict the passage of water. This prevents the buildup of damaging water pressure in the pores when concrete is frozen, and hence air-entrained concrete has greatly increased durability in outdoor locations exposed to freezing weather. Each of the first three types (I, II, or III) and types IS and IP are available as air-entrained. To signify this characteristic a letter A is added after the type. Thus, air-entrained pozzolan cement is known as type IP–A.

(4) *Other.* In addition to the types described above, there is white cement, waterproofed cement, and oil well cement. White cement is made from selected materials to prevent coloring, staining, or darkening of the finished concrete. Waterproofed cement has water-repellent material added. The finished and set concrete has a water-repellent action. Oil well cement is especially made to harden properly when used under high temperature in deep oil wells.

4–3. Water

Water plays an important part in the concrete mix. Its principal uses are to make the mix workable and to start the chemical reaction. Any material in the water which retards or changes the reaction is detrimental. A good rule of thumb is "if it's good enough to drink, it may be used for concrete."

a. Ordinary Water. The materials found in some types of water include organic compounds, oil, alkali, or acid. Each has its effect on the hydration process.

(1) *Organic material and oil.* These compounds tend to coat the aggregate and cement particles, and to prevent the full chemical action and adherence. The organic material may also react with the chemicals in the cement and create a weakened cementing action, thus contributing to deterioration and structural failure of the concrete.

(2) *Alkalies, acids, and sulfates.* These chemical impurities in the water tend to react with the chemicals in the cement. The result is an inadequate cementing and weakened concrete. Water must be free of these chemicals for use in concrete mixing.

b. Sea Water. The salts in sea water are normally thought of as being corrosive. However, sea water is used sometimes for concrete mixing with satisfactory results. A loss of 10 to 20 percent in compressive strength can be expected when using the same amount of sea water as fresh water. This can be compensated somewhat by reducing the water-cement ratio.

4–4. Aggregates

The aggregates commonly used for concrete are natural deposits of sand and gravel, where available. In some localities, the deposits are hard to obtain and large rocks must be crushed to form the aggregate. Crushed aggregate costs more to produce (in most cases) and will require more cement paste due to its shape. More care must be used in handling crushed aggregate to prevent poor mixtures and improper dispersion of the sizes through the finished concrete. At times, artificial aggregates such as blast furnace slag or especially burned clay are used.

a. Types of Aggregate. Aggregates are divided into two types: fine and coarse. When properly proportioned and mixed with cement these two groups will yield an almost voidless stone which is strong and durable. Aggregate must be equal to or better in strength and durability than the hardened cement to withstand the designed loads and the effects of weathering.

b. Fine Aggregate. Fine aggregate is defined as the material which will pass a No. 4 sieve and will be predominantly retained on a No. 200 sieve. To increase workability and for economy as reflected by use of less cement, the fine aggregate should have a rounded shape. The purpose of the fine aggregate is to fill the voids in the coarse aggregate and to act as a workability agent. This characteristic of workability is discussed more fully in the description of finished concrete (para 4–5*d*).

c. Coarse Aggregate. Coarse aggregate is a material which will pass the 3-inch screen and will be retained on the No. 4 sieve. As with fine aggregate, to increase workability and for economy as reflected by use of less cement, the coarse aggregate should have a rounded shape. Even though the definition seems to limit the size of coarse aggregate, other considerations must be accounted for.

(1) It can be readily seen that the more coarse aggregate, the more economical the mix. Larger pieces offer less surface area of the particles than an equivalent volume of small pieces. Use of the largest permissible maximum size of coarse aggregate permits a reduction in cement and water requirements.

(2) One restriction usually assigned to coarse aggregate is its maximum size. Large pieces can interlock and form arches or obstructions within a concrete form. This restricts the area below to a void, or at best, to become filled with the finer particles of sand and cement only. This is either a weakened area or a cement-sand concentration which does not leave the proper proportion to coat the rest of the aggregate. The maximum size of coarse aggregate must be *no larger than* the sizes given in table 4–1. Capacity of mixing equipment also may limit maximum aggregate size.

Table 4–1. Maximum Recommended Size of Coarse Aggregate

	Minimum dimension—inches			
Structure	*2½–5*	*6–11*	*12–29*	*30 or more*
Reinforced walls, beams and columns	½–¾	¾–1½	1½–3	1½–3
Unreinforced walls	¾	1½	3	6
Slabs, heavily reinforced	¾–1	1½	1½–3	1½–3
Slabs, lightly reinforced	¾–1½	1½–3	3	3–6

Note. Maximum size not to exceed one-fifth of minimum dimension of a wall or similar structure, one-third of slab thickness for horizontal slab, or three-fourths of minimum clear spacing between reinforcing bars.

d. Gradation. Gradation of aggregate refers to the amount of each size of particle used in the mix. A too large proportion of coarse aggregate leaves voids which require more cement paste to fill. This affects the economy of the mix. Too much fine aggregate, besides preventing good bonding, also increases the surface area which must be coated with cement paste. This will weaken the concrete. Good gradation results in a dense mass of concrete with a minimum volume of voids, an economical mix, and a strong structure. Optimum strength, watertightness, and durability in the hardened concrete require careful control of aggregate gradation.

e. Durability. Durability is the ability to resist the elements of weathering and the load pressures. Weak or easily crushed rock or other mineral particles that break down under the applied loads will cause changes in the internal stresses and a breakdown of the concrete. Rocks or mineral particles that are absorptive or susceptible to swelling when saturated will disintegrate when acted upon by different weather conditions. Freezing moisture causes expansion stresses which can easily rupture absorptive rocks. Radiant heat from the sun causing rock to swell followed by sudden cooling due to a shower and temperature drop causes shrinkage and a breakdown of some rocks. The aggregate must be chosen to withstand these forces of nature.

f. Deterioration. Deterioration of concrete in many cases can be traced to the aggregate. An excessive amount of organic material in or on the aggregate prevents the cement paste from forming an adequate bond with the aggregate particles. A large percentage of clay or fine silts adhering to the aggregate may prevent the cement paste from reaching the particles. This will result in a structurally weak concrete which is also susceptible to breakdown by weathering. Washing the aggregate to remove the silts, clays, and organic material will avoid this problem.

g. Chemical Composition. Chemical composition of the aggregate is also important. Any chemical reaction between aggregate and cement in the presence of water reduces the hardening and cementing processes. Any reduction in the amount of water-ement paste due to chemical reaction reduces the mount available to bond to the aggregate. This esult is similar to one caused by an insufficient mount of cement.

–5. Desirable Properties of Concrete

'o combine the ingredients correctly and to form the equired concrete, it is essential to know the properies of both the fluid or plastic mix and the hardened oncrete. The hardened concrete must have adequate trength (as required by the design of the strucure), be watertight, and be resistant to the deterirating effects of weather or wear. The quality and haracter of the hardened concrete is greatly influnced by the properties of the mix when plastic. To ttain optimum quality, the plastic mix must be uniorm, homogeneous, and workable. This permits placing the concrete without developing segregation, honeycombing, or other defects in filling the forms r in producing the desired smooth, hard surface.

a. Strength. This is the ability of the concrete to esist a load in compression, bending, or shear (secs. V and VI below). The desired design strength is btained by proportioning the mix of correctly graded aggregates, an adequate amount of cement o coat the surface area of the particles, and the proper amount of mixing water. The most important nfluencing factor on strength is the ratio of water o cement (referred to as the W/C ratio). For plastic and workable mixes, low values of the W/C ratio give higher strengths. Practice shows that 2½ gallons s the minimum amount of water that will adequately hydrate one sack (94 pounds) of cement. This amount of water is not sufficient to provide economically he needed plasticity and workability of the freshly mixed concrete. Hence additional water must be used, but the amount is held as low as consistent with workability requirements to produce the desired trength with economy of cement. Additional water will thin the paste and coat more particles. This inreases the "yield" from each sack and produces a more economical mix. Excessive amounts of water (too high a W/C ratio) weaken the paste by allowng the cement to harden while suspended in water without being in contact with the aggregate or other ement particles. In addition, much of this water evaporates and leaves holes or voids in the hardened concrete with a comparative reduction in strength. Minimum and maximum amounts of water are specfied to assure an economical mix with no loss in trength. This ranges from 4 gallons to 8 gallons per sack of cement.

b. Durability. The ability of concrete to resist the elements is called durability. The elements affecting concrete are wind, abrasion, freezing and thawing, wetting and drying, and the chemical action of salts. As the W/C ratio is increased (from 4 gallons per sack), more voids develop in the hardened concrete and more surface area is presented for the elements to act upon. This results in a less durable structure.

c. Watertightness. A well-mixed, well-proportioned concrete presents a solid surface to water penetration. Small surface voids permit some water to enter, but the water soon meets a solid mass which prevents further penetration. As the W/C ratio is increased, the excess water forms more holes which eventually connect to form channels into and through the structure. The result is a porosity which permits water to pass. For watertightness, 6 gallons of water or less per sack of cement will meet the specifications.

d. Workability. Workability is the relative ease or difficulty of placing and consolidating concrete. It is governed primarily by the amount of each aggregate in proportion to a given quantity of cement paste. As more aggregate is added to a given amount of paste, the mix becomes stiffer. The increased stiffness makes it more difficult to work the mix in the forms and around the reinforcing bars. The consistency needed depends upon the conditions under which the concrete must be placed and finished. Very dry and stiff mixes may be placed in many situations when high frequency vibration can be used to assist in consolidating and compacting the fresh concrete. In other situations, difficult placing conditions may require a very fluid mixture to fill narrow forms and to flow in between reinforcement.

e. Homogeneity. Concrete is a fluid mix combining particles of different sizes. The heavier particles tend to settle through the mix faster than the lighter particles. The result is layers of different consistency through the concrete and poor quality. When properly mixed and carefully handled, concrete particle separation (segregation) can be held to a minimum. The mix must have the proper proportion of cement-sand mortar to prevent the larger coarse aggregate particles from separating from the batch (segregation) during mixing and placing. Proper mixing and careful handling during placing will hold such segegation to a minimum. One handling operation causing particle separation is allowing the mix to drop (free fall) over a considerable distance. To minimize segregation, for drops in excess of 3 to 5 feet, bottom-dump buckets should be used to put concrete directly in place. See TM 5–742 for proper construction procedures.

f. Uniformity. Uniformity refers not only to a single batch of mix, but to all batches for an entire project. The same amount of each ingredient should be mixed into each batch or a nonuniform structure will result. The design criteria would not be met in all sections of the structure and failure of these sections could result. Proper supervision of each step in mixing and handling of batches will insure uniformity.

4–6. Curing

Concrete does not develop its full strength until the chemical process of curing called hydration is complete. Curing takes place over an extended period, the most critical portion of which is from the day of placement through the 10th day. The extent and rate of curing depends upon the presence of moisture and the correct temperature within the mix.

a. Temperature. The ideal temperatures for concrete work are between 55° F. and 70° F. Above this, rapid evaporation of moisture creates a problem. At lower temperatures, the curing or setting is delayed. Temperatures below 32° F. stop the hydration process. Since the chemical action gives off some heat, some method must be used to keep the heat within the structure during times of low temperatures. Cold weather construction may even require heating the ingredients or mix, and covering the emplaced concrete or providing a heated inclosure. In hot weather, extra care is required to prevent a high temperature rise and too rapid drying of the fresh concrete. Moistening the aggregate with cool water will lower the generated temperature. To keep the water as cool as possible, reflective white or aluminum paint is applied to water supply lines and storage tanks. On massive construction such as dams and heavy retaining walls, the mixing water is often cooled artificially or ice is substituted for part of the water. This ice must be melted by the time the concrete is fully mixed and is ready to leave the mixer. Cement replacement materials (such pozzolans as diatomaceous earth, pumicites, or fly ash) may be used to depress concrete temperature by reduction of heat of hydration in a structure. However, pozzolans vary widely and may have adverse effects on strength, air content, and durability if used in excessive amounts.

b. Moisture. Concrete curing depends upon chemical action in the presence of water. Any moisture lost during the process by seepage or evaporation prevents complete hydration and development of optimum strength and watertightness. Saturating the subgrade on which the concrete will be placed will delay, if not prevent, seepage from removing moisture. Wood forms should be thoroughly wetted if they have not been treated otherwise. Covering the concrete without marring the surface as soon as possible after finishing is one method to reduce evaporation. This covering may be some material such as straw, burlap, or plastic film, or it may be a chemical curing compound which is sprayed over the finished surface. After the initial set is attained, water can be applied directly to the surface to keep the hydration process in action. This water application can be part of the temperature control. The increase of concrete compressive strength with age is shown by the curves in figure 4-1. Note the long-time gain in strength th occurs when proper temperature and moisture co ditions are maintained.

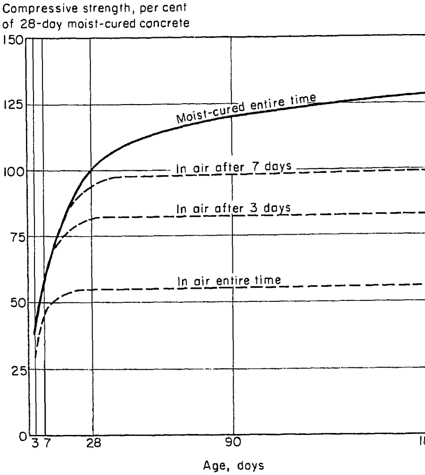

Figure 4-1. Increase of concrete compressive strength wi curing age.

4–7. Admixtures

Several chemical agents, or admixtures, are availab to improve workability, increase resistance to free ing and thawing, and compensate for inadequa curing time and conditions.

a. Accelerators. Sometimes, it is desirable to a celerate the hydration reaction. The result is a high early strength and a higher rate of heat productio This combination can be useful in winter operatior The addition of a chemical accelerator (general calcium chloride) to the mix will produce the desir conditions. The amount specified is 2 percent of tl weight of cement and rarely more than 3 percent. Tl main reaction with calcium chloride occurs with the first 3 days. The ultimate strength of concrete not affected by the use of this chemical.

b. Retarders. Retarders are used when excessive high heat and too rapid setting of concrete will pr vent full hydration. Many materials will retard t setting of concrete. Basically, they are types of fat acids, or starches and sugars.

c. Workability Agents or Plasticizers. The wor ability of concrete is governed by the amount of a gregate in a mix. Where reduction of aggregate (increase in cement) is impractical, workability increased by adding a plasticizer. Air-entraini agents, when used, are plasticizers. Other substan

s. Calcium chloride is also an accelerator (a ove). Lime increases the cementing properties of ment, as do pozzolans combined with lime. Fly ash very inexpensive as compared to cement and is used a partial replacement (up to as much as 50 percent) the cement. It changes both the plastic and the rdened properties of concrete. Fly ash improves orkability and reduces segregation, bleeding, and e heat of hydration. The concrete will not be as atertight as a cement-only concrete, nor will it have much initial strength. Additional tests may have be made to determine when to remove the forms. s final strength, however, will be as great as a ment-only concrete.

d. Densifiers. Dense concrete is required in some pes of construction, such as in prestressed strucres. To achieve this density, the cement particles e separated evenly throughout the mix, or at least evented from attaching to each other (flocculatg). A detergent admixture will disperse the partis individually and will create a more uniform ste. These admixtures also reduce the formation a cement gel which expands at the early stages of dration and pushes the particles apart thus in-

e. Waterproofing Agents. Watertightness (para 4–5*c*) can be controlled to a great extent by lowering the W/C ratio. This may not be practical, or even with a fairly low W/C ratio, capillaries may still form through the concrete. Densifying or using an accelerator like calcium chloride will improve the watertightness.

f. Air-Entraining Agents. The greatest improvement in watertightness and resistance to deterioration under freezing and thawing is obtained by incorporating 4 to 6 percent by volume of entrained air into the mix. Workability of fresh concrete is enhanced by such entrained air. Air-entraining cement contains the necessary admixture. Soaps, oils, butyl stearate, some of the fine pozzolans, and several proprietary compounds are available for use as air-entraining admixtures with ordinary cements. These agents minimize the formation of capillaries and plug the tiny holes with a water-repellent or sealing material. They provide very small, uniformly-spaced, discrete air voids that prevent the buildup of damaging pressures from the expansion into ice of freezing water.

Section II. AGGREGATE TESTING

-8. Sampling

order for the aggregate tests to be worthwhile, e sample secured for testing must be representative the supply of aggregates. If possible, samples ould represent the processed and ready-for-use gregate. Sufficient samples should be taken from e processing plant discharge to represent the marial in the stockpile. The sample should consist of least four times as much material as is needed for e tests, and should be reduced to the size desired r testing by quartering.

a. Stockpile Samples. Stockpile samples should be ken at or near the base, at about the middle, and the top of the pile. Three or four such samples ould be taken and recombined. A board shoved into e pile just above the point of sampling will prevent e material above the sampling point from falling sifting into the sample.

b. Pit Samples. Unprocessed sources of sand and avel should be sampled by channeling exposed ces or in pits, if exposures are not available. Care ould be taken to insure that the samples include ly those materials which are below the overburden strip zone.

-9. Gradation

sieve analysis indicates whether an aggregate is arse or fine and whether it is evenly or unequally stributed between the larger and smaller sizes. This information is useful in deciding whether the aggregate will make good concrete. See table 4–2 for acceptable gradation limits.

Table 4–2. Desirable Gradation for Aggregates in Concrete

a. Coarse aggregate.

Sieve size in.	*Percent passing indicated sieve* *3*	*2½*	*2* *No. 357*	*1½* *No. 467*	*1* *No. 57*	*¾* *No. 67*	*½* *No. 7*
3	100						
2½	88	100					
2	76	86	100				
1½	61	69	81	100			
1	44	49	58	72	100		
¾	33	38	44	55	76	100	
½	21	24	28	35	48	63	100
⅜	14	16	18	23	32	41	65
No. 4	------	------	------	------	------	------	------

b. Fine aggregate.

Sieve size U.S. Standard	*Percentages by weight passing*
4	95–100
8*	80–100
10	75–95
16*	50–85
20	40–75
30*	25–60
40	20–50
50*	10–30
60	10–25
100	2–10

*ASTM C 33 specifies U.S. Standard sieve sizes No. 8, 16, 30, and 50 instead of No. 10, 20, 40 and 60. Both ranges are provided for convenience.

1½-inch sieve, 58 percent passing the 1-inch sieve, etc. Acceptable aggregates should be near to these gradation limits, say within ±5 percent of the figures given in the table.

a. Apparatus, Test Procedure, and Calculations. The apparatus, test procedure, and calculations required to determine the gradation of aggregates for portland cement concrete are the same as explained in paragraph 3–20*b*, except that the No. 4 sieve is taken as the dividing line between fine and coarse aggregate. The size of sample required in the sieve analysis of fine aggregate is 300 grams. The result of this test is a gradation curve for the aggregate concerned.

b. Material Finer Than No. 200 Sieve. In deciding whether an aggregate will make good concrete, it is important to know the amount of material passing the No. 200 sieve. The apparatus, test procedure, and calculations are described in paragraph 4–11*a*. Fine material, not to exceed 3 to 4 percent of the total aggregate weight, is not harmful to concrete. For some purposes, a small amount of such fines may improve the workability.

c. Fineness Modulus. The fineness modulus is an empirical factor that gives a relative measure of the proportions of fine and coarse particles in a fine aggregate. The fineness modulus does not give any idea of the graduation of the material although the process is similar. A 500 gram sample of sand is sieved through a series of sieves (No. 4, 8*, 16*, 30*, 50*, and 100). The weight retained on each sieve is converted into a cumulative weight and a cumulative percentage retained, starting with the No. 4 sieve. The sum of the six percentages is divided by 100 and results in the fineness modulus. Fineness modulus determines the volume of coarse aggregate per unit volume of concrete when being considered as a factor in the mixture. The fineness modulus values are interpreted as follows:

Fine sand=2.20–2.60
Medium sand=2.60–2.90
Coarse sand=2.90–3.20

4–10. Specific Gravity, Absorption, and Surface Moisture

These tests must be performed on the aggregates before the necessary calculations can be made to design the concrete mixture. In aggregates used in portland cement concrete, measurements are made of the bulk specific gravity with the aggregates in a saturated, surface-dry condition. Specific gravity determinations are thus based upon determinations of the total volume occupied by the aggregate particles, includ-

calculation of the amount of mixing water to be used in concrete mixture.

a. Apparatus. The following equipment is necessary to perform tests for bulk specific gravity, percent absorption, and surface moisture:

Chapman flask
Concrete test apparatus, Dunagan
Cone pycnometer
Fan, electric
Gloves, lineman's
Mold, cone, water-absorption
Oven, electric
Pan, bake
Rod, tamping, steel, flat-head
Scoop, kitchen
Shovel, square-point
Spatula, 4-inch

The water-absorption cone and tamping rod are shown in figure 4–2. The Dunagan concrete test apparatus is illustrated in figure 3–10.

b. The bulk specific gravity of coarse aggregate in a saturated, surface-dry condition is determined as follows:

(1) The Dunagan concrete test apparatus is assembled.

(2) A representative sample weighing approximately 5,000 grams is secured.

(3) The sample is immersed in water and allowed to soak for not less than 24 hours.

(4) The sample is removed from the water and rolled in a large, absorbent cloth until all visible films of water are removed, although the surfaces of the particles will still appear to be damp. The larger fragments may be wiped individually.

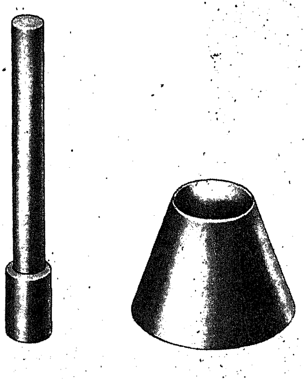

Figure 4–2. Water-absorption cone and tamping rod.

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*See note in paragraph 2–24.

face-dry aggregate is weighed out. To do this, add a 2 kg slotted weight to the left hanger and pour the sample into a scoop on the right-hand pan until balanced. Remove the 2 kg weight and pour the sample into the bucket. Place the spare scoop, clean and dry, on the right-hand pan and balance with weights. This weight is the immersed weight.

(6) The bulk specific gravity is calculated as follows:

Bulk specific gravity (saturated, surface-dry conditions) =

$$\frac{2{,}000}{2{,}000\text{—immersed weight}}$$

c. The test procedure for determing the *bulk specific gravity of fine aggregate* in a saturated, surface-dry condition is as follows:

(1) The Dunagan concrete test apparatus is assembled.

(2) A representative sample weighing approximately 3,000 grams is secured.

(3) The sample is immersed in water and allowed to soak for 24 hours.

(4) At the end of the 24-hour soaking period, the sample is spread on a flat surface and stirred frequently so as to obtain uniform drying. Drying is continued until the sample approaches a moisture-free surface condition.

(5) The conical mold is placed large end down on a smooth surface and filled loosely with the aggregate, after which the surface of the aggregate is tamped lightly 25 times with the metal rod.

(6) The conical mold is lifted vertically from the sand. If free moisture is present, the cone of fine aggregate will retain its shape.

(7) Drying is continued, accompanied by constant stirring, and the cone tests (5) and (6) repeated at frequent intervals until the cone of fine aggregate slumps upon removal of the mold. This indicates that the fine aggregate has reached a surface-dry condition.

(8) Exactly 1,000 grams of the saturated, surface-dry sample is weighed out. This may be conveniently done by placing a 1-kg slotted weight on the left hanger and the sample in a scoop in the right-hand side.

(9) Using the Dunagan apparatus, the weight in water of this 1,000 grams of saturated, surface-dry sample is obtained immediately. This is recorded as "immersed weight."

(10) The specific gravity is calculated as follows:

Bulk specific gravity (saturated, surface-dry)

$$=\frac{1{,}000}{1{,}000\text{—immersed weight}}$$

absorption of either coarse or fine aggregate is as follows:

(1) Representative samples of the fine and coarse aggregates are secured. The sample of fine aggregate should weigh approximately 500 grams. Coarse aggregate which contains no pieces larger than 1½ inches should be represented by at least 3,000 grams, while coarse aggregate which contains pieces larger than 1½ inches should be represented by at least 5,000 grams.

(2) The samples of fine aggregate and coarse aggregate are immersed in water. The fine aggregate is allowed to soak for 2 hours and the coarse aggregate for at least 24 hours.

(3) The samples are dried to a saturated, surface-dry condition, following the methods described in the specific gravity test procedures (*c* above).

(4) The weights of the samples in a saturated, surface-dry condition are obtained and recorded.

(5) The samples are dried to constant weight in the electric oven, maintained at a temperature between 100° C. and 110° C. (212° F. and 230° F.).

(6) The oven-dry weights of the samples are obtained and recorded.

(7) The percent absorption is calculated by the use of the following formula:

$$X=\frac{(SSD—D)}{D}\ (100)$$

Where:

x=percent absorption by the aggregate,
SSD=weight of saturated, surface-dry sample, grams, and
D=oven-dry weight of sample, grams.

(8) The percent absorption represents the moisture content (oven-dry basis) of the fine aggregate when the aggregate is in a saturated, surface-dry condition.

e. Surface moisture is the water which is present in the fine aggregate, over the above that which corresponds to a saturated, surface-dry condition. This water will become part of the mixing water when the fine aggregate is used in making concrete. The amount of mixing water used must be corrected to allow for its presence. The test described in this paragraph is used only if the total moisture content of the aggregate is greater than the total moisture content corresponding to a saturated, surface-dry condition. By definition, the total moisture content of the aggregate in a saturated, surface-dry condition is the percent absorption. If the bulk specific gravity (saturated, surface-dry) has not been determined previously, this determination is made, using the procedure given in *c* above. The test procedure for determining the amount of surface moisture present in fine aggregate is as follows:

(1) Exactly 1,000 grams of the sample on which the surface moisture determination is desired is weighed out. The sample will be representative of the condition of the fine aggregate at the time the test is made.

(2) Using the Dunagan apparatus, the weight of this 1,000-gram sample immersed in water is obtained. This weight is recorded as (W_2).

(3) The amount of surface moisture is calculated from the following formula:

$$SM=(W_1-W_2)\,\frac{Sp\ Gr}{Sp\ Gr-1}$$

Where:

SM = grams of surface moisture in sample
W_1 = weight of a 1,000-gram sample of saturated surface-dry material immersed in water (taken from data for bulk specific gravity determination, (c(9) above), grams
W_2 = immersed weight of the 1,000-gram test, sample, (2) above, grams, and
Sp Gr = bulk specific gravity (saturated, surface-dry).

(4) The surface moisture expressed as a percent (P_1) of the saturated surface dry weight of fine aggregate is—

$$P_1=\frac{SM}{1000-SM}\,(100)$$

(5) The surface moisture expressed as a percent (P_2) of the weight of wet fine aggregate is—

$$P_2=\frac{SM}{1000}\,(100)$$

f. An *alternative determination* of surface moisture on fine aggregate is obtained by drying completely a sample having known weight of wet fine aggregate, and calculating total moisture as:

$$\text{Total moisture}=\frac{\text{wet weight}-\text{dry weight}}{\text{dry weight}}\,(100)$$

The absorption, expressed as percent of dry weight (*d*(7) above) is subtracted from this total moisture to give the surface moisture in percent of dry weight. Drying of this sample is accomplished in an oven maintained at a temperature between 100° C. and 110° C. (212° F. and 230° F.) or by placing the sample in a metal pan, pouring alcohol over it, and burning off the alcohol to evaporate the water.

4–11. Tests for Impurities

The quality of the aggregate is another important onsideration. The presence of organic material, exessive quantities of clay or silt, and shale or other water-absorbing particles can be detrimental to the oncrete strength, watertightness, and durability. Recommended permissible limits for deleterious substances in fine and coarse aggregates are given in tables 4–3 and 4–4.

Table 4–3. Limits for Deleterious Substances in Fine Aggregate for Concrete

Item	*Maximum, percent by weight of total sample*
Clay lumps	1.0
Material finer than No. 200 sieve:	
Concrete subject to abrasion	[a] 3.0
All other concrete	[a] 5.0
Coal and lignite:	
Where surface appearance of concrete is of importance	0.5
All other concrete	1.0

[a] In the case of manufactured sand, if the material finer than the No. 200 sieve consists of the dust of fracture, essentially free from clay or shale, these limits may be increased to 5 and 7 percent, respectively.

Table 4–4. Limits for Deleterious Substances in Coarse Aggregate for Concrete

Item	*Maximum, percent by weight of total sample*
Clay lumps	0.25
Soft particles	5.0
Chert as an impurity[a] that will disintegrate in five cycles of the soundness test, or 50 cycles of freezing and thawing (0° F. to 40° F.)[b]; or that has a specific gravity, saturated-surface-dry, or less than 2.35:	
Severe exposure	1.0
Mild exposure	5.0
Material finer than No. 200 sieve	[c] 1.0
Coal and lignite:	
Where surface appearance of concrete is of importance	0.5
All other concrete	1.0

[a] These limitations apply only to aggregates in which chert appears as an impurity. They are not applicable to gravels that are predominantly chert. Limitations on soundness of such aggregates must be based on service records in the environment in which they are used.

[b] Disintegration is considered to be actual splitting or breaking as determined by visual examination.

[c] In the case of crushed aggregates, if the material finer than the No. 200 sieve consists of the dust of fracture, essentially free from clay or shale, this percentage may be increased to 1.5.

a. Test for Material Finer Than No. 200 Sieve. The presence of fine particles of soil and clay can affect the concrete in two ways. The added surface area of the fine particles picks up the cement paste and reduces the amount available to bind and hold the aggregate. The small particles also tend to float up to the surface when the concrete is finished (especially when wet mixes are used). This results in a surface covered by hair cracks and a tendency for the fines to dust off when dry.

(1) *Apparatus.* Two sieves (No. 16* and No. 200) and a container large enough to held the sample covered with water are required. The container

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*See note in paragraph 2–24.

hould allow vigorous shaking without loss of either sample or water.

(2) *Procdure.*

(*a*) Select a representative sample as follows:

Nominal maximum sieve size	*Approx. minimum weight (kg)*
No. 4	0.5
¾-in.	2.5
1½-in. or larger	5.0

(*b*) Oven-dry the sample at 110° C. (230° F.) and weigh it.

(*c*) Place the dry material into the container and cover it with water.

(*d*) Agitate the material vigorously to bring the fine portion into suspension.

(*e*) Decant the water and run it through the sieves. Any material retained on the sieves is returned to the sample container.

(*f*) Repeat the operation until the water runs clear.

(*g*) Oven dry the sample at 110° C. (230° F.), to a constant weight.

(*h*) Weigh the dry sample and record the weight.

(3) *Computations.* The percent fines can now be calculated as follows:

$$\text{Percent fines} = \frac{\text{original dry weight—dry weight after washing}}{\text{original dry weight}} \times 100$$

(4) *Expedient test.* The method described above is accurate but time consuming. For a quick and approximately correct determination, a 1000 g representative sample is obtained. About 2 inches is placed into a quart mason jar which is then filled three-fourths full with water. The mixture is shaken vigorously and then allowed to stand for 1 hour. The silt and clay layer will form on top of the sand. If the layer is more than ⅛ inch thick, the material has more than 3 percent fines and should be washed before using.

b. Test for Clay Lumps. This test is performed on the material remainig after completion of the determination of material finer than the No. 200 sieve. The cleaned material is oven-dried at 110° C. (230° F.), weighed, and separated into sample sizes. For fine aggregate, a 100 gram sample of material coarser than the No. 16 sieve is required. Coarse aggregates are separated into four size ranges as follows: No. 4 to ⅜ inch; ⅜ inch to ¾ inch; ¾ inch to 1½ inch; and over 1½ inch. Minimum sample weights for these four size ranges are: 1000 grams, 2000 grams, 3000 grams, and 5000 grams respectively. The tests are performed as follows:

(1) Spread the sample in a thin layer on the bottom of a flat pan and examine it for clay lumps. Break up all such particles by crushing them between the fingers. Remove the residue from the broken clay lumps by sieving dry over the appropriate one of the following sieves: fine aggregate—No. 20; No. 4 to ¾ inch—No. 8; ⅜ to ¾ inch—No. 4; ¾ to 1½ inch—No. 4; over 1½ inch—No. 4. Weigh the residue retained on the sieve (W_2).

(2) Calculate the percentage of clay lumps as follows:

$$\%C = \frac{W_1 - W_2}{W_1}\,(100)$$

Where:

$\%C$ = percentage of clay lumps

W_1 = weight of sample

W_2 = weight of residue after clay lumps are removed

c. Test for Undersirable Lightweight Material. Soft, laminated pieces of aggregate such as chert or shale are detrimental to concrete. Coal and lignite also are harmful and are distinguished from other lightweight materials by the brownish-black or black color of the particles. Visual examination of the coarse aggregate will often show these minerals. The amount of these minerals in an aggregate can be determined by immersing the aggregate in a liquid with a specific gravity that will allow the shale and other light particles to float, and the heavier particles to sink. Test procedures are as follows:

(1) Place a portion of the sample, dried to constant weight, into a container partly filled with a heavy liquid. A saturated solution of zinc chloride at room temperature 26.5° C. (78° F.) has a specific gravity of 1.95. This or any other liquid with this specific gravity can be used.

(2) Agitate the mixture five times.

(3) Skim off the lighter particles.

(4) Separate the liquid and pebbles and repeat the operation.

(5) When all the undesirable lightweight material has been removed, wash, dry, and weigh the remaining material. The difference between the final weight and the original dried weight (expressed as a percentage of the total weight) represents the undesirable material in the aggregate.

d. Color Test for Organic Matter. Any sand that gives a color darker than the standard of this test probably contains an excess of organic matter which will give trouble in concrete. It is possible that the organic matter can be removed by washing; if not, better sand should be obtained. If neither of these things can be done, it will be necessary to use a lower water cement ratio and control the concrete production carefully in order to obtain the desired strength.

(1) *Apparatus.* The following items are required for the conduct of this test:

Acid, tannic
Alcohol, ethyl
Bottle, prescription, 12 ounce, graduated
Cylinder, 100 cc, graduated
Plate, color
Sodium hydroxide, caustic soda

This apparatus is shown in figure 4–3.

(2) Test procedure. A reference standard color solution is prepared by adding 2.5 mililiters of 2-percent solution of tannic acid in 10 percent alcohol to 97.5 milliliters of a 3-percent sodium hydroxide solution. The 3-percent sodium hydroxide solution is made by dissolving 1 ounce of sodium hydroxide in enough water to make 32 fluid ounces. The 2 percent tannic acid solution is made by adding 10 milliliters of 190 proof alcohol and 2 grams of tannic acid powder to 90 milliliters of water. This solution is poured into a 12 ounce bottle, stoppered, shaken vigorously, and allowed to stand for 24 hours.

(*a*) A representative sample of about 500 grams of the material to be tested is obtained.

(*b*) A 12-ounce graduated bottle is filled to the 4½ ounce mark with the sample to be tested. A 3-percent solution of sodium hydroxide in water is added until the volume of sand and water after shaking is 7 liquid ounces. The bottle is then stoppered, shaken vigorously, and allowed to stand for 24 hours.

(*c*) After the standard color solution and the test sample have been allowed to stand 24 hours, the color of the referenced solution is compared with the color of the clear liquid above the test sample.

(3) *Evaluation of results.* If the liquid above the sample is darker than the standard color solution, the sand may contain organic impurities which will reduce the strength of the concrete in which the sand is used.

(4) *Alternate test procedure.* After the test sample has been prepared as described above and the sample set aside for 24 hours, the color of the liquid above the sample may be compared with the colors given in the standard color plate. A decision may then be made as to whether the sand contains an excessive amount of organic matter.

4–12. Test for Soundness

Soundness is the property of resisting disintegration. Some aggregates disintegrate when they absorb water and are exposed to a freezing and thawing cycle. Others containing shale can disintegrate under load pressures. The following test will indicate the presence of these deleterious materials.

a. Freeze-Thaw Test. In one form of this test, the aggregate is saturated with water and subjected to freezing and thawing cycles. The amount of breakage is measured to evaluate the soundness of the material. These tests are used primarily for research into the durability of concrete and aggregates.

b. Salt Test. The freeze-thaw operation requires considerable equipment and has generally been replaced by a simpler one that requires less equipment. This simple test uses solutions of special salts in which the aggregate is immersed and saturated. The crystals of these salts are permitted to grow and create a disruption similar to freezing water. The salts used are sodium sulfate (glauber salt) and magnesium sulfate (epsom salt). This test is performed as follows:

(1) Secure a representative sample. The sample must be separated on the sieves listed in table 4–5 until the minimum amount, as indicated, is retained on each sieve. If there is an indication that there is less than 5 percent of any sieved size in the entire sample, that size need not be tested for soundness. Instead, the average of the next two immediate sizes can be assumed to be representative of the nontested size.

Table 4–5. Minimum Amounts of Material for Aggregate Soundness Test

Sieve size	*Minimum amount for testing*			
Fine aggregate:				
No. 50	100 g.			
No. 30	100 g.			
No. 16	100 g.			
No. 8	100 g.			
No. 4	100 g.			
Coarse aggregate (after all sizes finer than No. 4 are removed):				
No. 4	300 g.			
⅜ in	1000 g.	⅜	½ in	33%
		½ in	¾ in	67%
¾ in	1500 g.	¾ in	1 in	33%
		1 in	1½ in	67%
1½ in	3000 g.	1½ in	2 in	50%
		2 in	2½ in	50%
Larger sizes (1-inch spread in sieves sizes)	3000 g.			

(2) Immerse the aggregate fractions in saturated solutions of the specified salts for a period of 16 to 18 hours.

(3) Oven dry the aggregate at a temperature of 105° to 110° C. (221° F. to 230° F.).

(4) Repeat the cycle as specified by the engineer in charge (generally 5 cycles are sufficient).

(5) Wash the fractions until all the salt is removed. Adding 20–30 g of barium chloride to the wash water will cause a precipitate to form if any salt is left. Continue washing until no precipitate is seen.

(6) Dry the washed fractions to a constant weight and place each one, in turn, on its designated sieve which is held over a pan. The amount passing the sieve is weighed and is called the "loss". It represents the particles that are chipped off during the test.

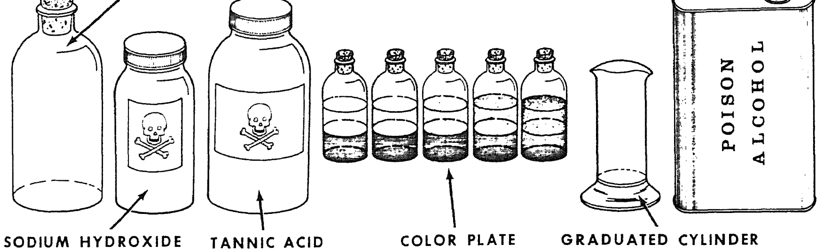

Figure 4–3. Apparatus, color test for organic matter.

(7) Convert the loss of each fraction into a percentage of the total sample weight.

(8) The report shall include the following information:

(*a*) Total weighted loss for the sample.

(*b*) For particles larger than ¾ inch, the number of particles before and after the test.

(9) The permissible limits are as follows:

(*a*) Sand—Total weighted loss not greater than 10 percent if sodium sulfate is used or 15 percent if magnesium sulfate is used.

(*b*) Coarse aggregate—Total weighted loss not greater than 12 percent if sodium sulfate is used or 18 percent if magnesium sulfate is used.

(*c*) Aggregates failing to meet these requirements are rejected unless they pass an actual freeze-thaw test.

(10) *Example:*

Fraction tested (passing No. 30—retained No. 50)

Weight at start	100.0 g
Weight at end	95.8 g
Loss	4.2 g

$$\text{Percentage loss of fraction} = \frac{4.2}{100} \times 100 = 4.2 \text{ percent}$$

Sieve analysis indicated that this fraction is 26 percent of the total sample. Adjusting fraction loss to total sample:

Percentage loss (No. 30—No. 50 fraction) of total $= 4.2 \times 26 = 1.09$ percent. This result signifies that 1.09 percent of the sand sample is lost in the No. 30 to No. 50 range. To complete the procedure, all size fraction losses (in percent of total) are added and reported as the soundness test weighted loss.

Section III. IDENTIFICATION OF CEMENTS

4–13. Introduction

The positive identification of cement, because of the very wide variety of related or similar-appearing materials, requires a complete chemical analysis and physical tests. The different types of cement are discussed in paragraph 4–2.

a. Every effort should be made to identify the unknown material directly by obtaining and, if necessary, translating, all labels, tags, shipping documents, manufacturing sheets, and all other papers which may contain applicable information. When this does not produce results, the simple procedures outlined in the following paragraphs generally will supply enough information to permit a tentative, if not conclusive, identification.

b. The following items will be required for some of the described tests:

Kerosene
Balance—2000 gram
Graduate, glass—1000 ml
Volumetric flask—500 ml
Small metal funnel
Pans or evaporating dishes
Square glass plates—4″ x 4″
Spatula—4 inch

4–14. Tests

The tests given below are made to identify whether the material is a cement, and then to attempt an identification of its type.

a. Hardening Test. A small sample of the material is selected and mixed with enough water to make a plastic paste of a consistency similar to that generally used in cement mortars. It is then molded into a pat about 3 inches in diameter and ¾ inch thick. The paste should be observed several times an hour to determine whether or not the paste is setting (hardening). The cement has attained a final set when the surface is hard enough to be unmarked when a pencil point or fingernail is pressed against it with moderate force. If it sets within 1 to 10 hours, the material is probably a cement.

b. Color. If it has been fairly welll established that the material in question is a cement, color may serve as a means of further classification. If the material is gray, it is likely to be a portland cement; if brownish gray, it may be a natural cement; if black, an aluminous cement; and if white, it probably is hydraulic lime, plaster, or possibly white portland cement.

c. Specific Gravity. The approximate specific gravity may be determined and used to provide an additional clue to the identity of the material. The approximate specific gravities of several types of cement are shown below, and may be determined by the outlined procedure.

(1) *Typical values.*

Specific gravity	*Probable material*
More than 3.3	Iron-ore cement
3.0 to 3.3	Portland cement
2.7 to 3.0	Natural cement
2.2 to 2.4	Plaster of paris

(2) *Calibration of flask.* A calibrated 500-ml. flask is accurately weighed in a dry condition. The flask is filled with water exactly to the calibration mark. The weight of the flask and water is determined. The water is poured out and the flask thoroughly dried. The flask is then filled to the calibration mark with kerosene and the weight determined. In expedient conditions, light fuel oil may be substituted for the kerosene.

(3) *Test procedure.* The weight of a sample (about 500 grams) of the material to be identified is determined as accurately as possible. The sample is placed in the calibrated, clean, dry flask and enough kerosene poured into the flask to cover it. The mixture is stirred thoroughly to remove all the air. Any air that is not removed by stirring will tend to give too low a value for specific gravity. The flask is filled to the calibration mark with kerosene, and the weight of the flask, sample, and kerosene determined. All weighings should be done at approximately the same temperature.

(4) *Calculations.* The approximate specific gravity of the cement may be calculated by use of the following formula: Specific gravity=

$$\left(\frac{A}{(D+A)-E}\right)\left(\frac{D-B}{C-B}\right)$$

Where:

A=weight of the sample
B=weight of the flask, empty
C=weight of the flask with water
D=weight of the flask with kerosene
E=weight of the flask, with sample and kerosene.

All weights normally would be in grams.

4-15. Air-Entraining Cement

To determine whether or not a given material contains an air-entraining agent, a sample is placed in a glass cylinder to a depth of about 1 inch. Water is added to a depth of about 6 inches and the cylinder and its contents shaken vigorously. If a considerable volume of stable, persistent foam forms on the surface, the cement probably contains an air-entraining agent (para 4-7*f*).

4-16. High-Early Strength Cement

High-early strength cement (Type III, para 4-2*c*) may be recognized by making a batch of concrete using the unknown cement in parallel with a similar batch which uses a known cement. Concrete which contains high-early strength cement will usually harden in less time than concrete which contains regular portland cement. High-early strength cement concrete, if molded into standard concrete beams and tested after 3 days for flexural strength, should have a modulus of rupture more than 150 pounds per square inch higher than similar specimens containing regular portland cement concrete.

Section IV. PROPORTIONING OF MIXES

4-17. Discussion

Cement, water, and fine and coarse aggregates are the basic materials used in the production of concrete. Certain admixtures (para 4-7) are used occasionally to meet special requirements. Design of a concrete mix consists of determining the correct amount of each material to provide a mixture of concrete having the necessary consistency or workability in the freshly-mixed condition and having the desired strength and durability characteristics in the hardened condition. The workability required depends upon mixing, placing, and finishing conditions of the job and is specific in terms of the slump test (para 4-23). Strength requirements, specified by the engineer for the particular construction, and exposure conditions determine the correct water-cement ratio to be used. Gradation of the fine and coarse aggregates is the factor that controls the proportion of each in the total amount of aggregate. The maximum size of coarse aggregate particle is limited by the dimensions of the structure being built (para 4-4*c*). Coarse aggregate size and workability requirements establish the amount of water needed, and hence the water-cement ratio fixes the required quantity of cement in (the cement factor, in 94 pound bags per cubic yard of concrete) is specified. The care and intelligence used in proportioning these ingredients are evident in the finished product; poor concrete or quality concrete. There are two methods of proportioning available to the military engineer, the book method and the trial batch method.

a. Book Method of Mix Proportioning. Concrete may be made with a wide variation in quality. The considerations which the proportioning must follow are given under the desirable properties of concrete (para 4–5). The selections of the correct water-cement ratio, the allowable slump, and the type and size of aggregate are all part of the design function. Having all of the above information, the design engineer can use prepared tables to establish the weights of materials to be used and the expected yield for a one sack of cement batch. Mixes so proportioned may require modifications in the field to achieve the desired quality. This method is discussed further in paragraph 4–19.

b. Trial Batch Method of Mix Proportioning. The trial batch method is a simple field method which incorporates many of the best features of several other methods, and it may be used under almost any circumstances. The trial batch method is capable of producing, with a minimum of tests, a satisfactory mix using the materials which will be used in the project. Such trial mixes are relatively small batches made with laboratory precision to establish the proportions of materials that produce a mix of required workability characteristics, density, and economy. The minimum cement content or water-cement ratio, maximum size of aggregate, air content, and consistency (slump) are selected to meet job requirements. Then mixes are made in which the aggregate proportions are adjusted by trial until the most economical proportions that provide a dense, workable mixture are found. The final quantities of materials for the field mixes are calculated by using a ratio from the quantities used in the selected trial batch. See paragraph 4–20 for detailed discussion.

The strength required of the concrete, the exposure conditions, and the kind and size of members to be constructed must be known in order to design a mix having the needed characteristics. Selection of the water-cement ratio fixes, to a large extent, the quality of the hardened concrete. Total quantity of cement and water then is found from the workability requirements. Finally, aggregate proportions are determined.

a. Water-Cement Ratio. Recommended values of water-cement ratio for various strength and exposure requirements are given in tables 4–6 and 4–7, respectively. To attain the desired compressive strength of concrete, the water-cement ratio should not exceed the value shown in table 4–6. The type of structure and degree of exposure also set maximum limits on water-cement ratio as given in table 4–7. The smaller ratio derived from these two tables governs the mix design.

Table 4–6. Compressive Strength of Concrete for Various Water-Cement Ratios

Water-cement ratio, gal per bag of cement	*Probable compressive strength at 28 days, p.s.i.*	
	Non-air-entrained concrete	*Air-entrained concrete*
4	6, 000	4, 800
5	5, 000	4, 000
6	4, 000	3, 200
7	3, 200	2, 600
8	2, 500	2, 000
9	2, 000	1, 600

b. Consistency. Workability or consistency of freshly-mixed concrete is measured by the slump test (para 4–23). Recommended slumps (in inches) for various types of construction are given in table 4–8. The stiffness of the mix is inversely proportional to slump. In the absence of any other criteria, concrete should be placed as "stiff" as possible, yet maintaining a homogeneous, voidless mass.

c. Water Content. The amount of mixing water required for the desired slump depends upon maximum size of aggregate. In table 4–9 are given approximate water requirements in gallons per cubic yard of concrete for concrete mixes made from different sizes of aggregate for ranges in slump.

*Table 4-7. Maximum Pemissible Water-Cement Ratios for Different Types of Structures and Degrees of Exposure**

Type of structures	Exposure conditions**					
	Severe wide range in temperature or frequent alternations of freezing and thawing (air-entrained concrete only) (gallons / sack)			Mild temperature rarely below freezing, or rainy, or arid (gallons / sack)		
	In air	At water line or within range of fluctuating water level or spray		In air	At water line or within range of fluctuating water level or spray	
		In fresh water	In sea water or in contact with sulfates†		In fresh water	In sea water or in contact with sulfates†
A. Thin sections such as reinforced piles and pipe	5.5	5	4.5	6	5.5	4.5
B. Bridge decks	5	5	4.5	5.5	5.5	5
C. Thin sections such as railings, curbs, sills, ledges, ornamental or architectural concrete, and all sections with less than 1-in. concrete cover over reinforcement	5.5	-	-	6	5.5	-
D. Moderate sections, such as retaining walls, abutments, piers, girders, beams	6	5.5	5	††	6	5
E. Exterior portions of heavy (mass) sections	6.5	5.5	5	††	6	5
F. Concrete deposited by tremie under water	-	5	5	-	5	5
G. Concrete slabs laid on the ground	6	-	-	††	-	-
H. Pavements	5.5	-	-	6	-	-
I. Concrete protected from the weather, interiors of buildings, concrete below ground	††	-	-	††	-	-
J. Concrete which will later be protected by enclosure or backfill but which may be exposed to freezing and thawing for several years before such protection is offered	6	-	-	††	-	-

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*Adapted from Recommended Practice for Selecting Proportions for Concrete (ACI 613-54).

** Air-entrained concrete should be used under all conditions involving severe exposure and may be used under mild exposure conditions to improve workability of the mixture.

†Soil or groundwater containing sulfate concentrations of more than 0.2 percent. For moderate sulfate resistance, the tricalcium aluminate content of the cement should be limited to 8 percent, and for high-sulfate resistance to 5 percent. At equal cement contents, air-entrained concrete is significantly more resistant to sulfate attack than non-air-entrained concrete.

††Water-cement ratio should be selected on basis of strength and workability requirements, but minimum cement content should not be less than 470 lb per cubic yard.

Types of construction	*Maximum*	*Minimum*
Reinforced foundation walls and footings	5	2
Plain footings, caissons, and substructure walls.	4	1
Reinforced slabs, beams, and walls	6	3
Building columns	6	3
Pavements	3	2
Heavy mass construction	3	2
Bridge decks	4	3
Sidewalks, driveways, and slabs on ground	6	3

*When high-frequency vibrators are used, the values may be decreased approximately one-third; in no case should the slump exceed 6 inches.

water requirement (table 4–9) and water-cement ratio (tables 4–6 and 4–7). Under field conditions where inspection and careful control of concrete quality are difficult, a minimum cement content should be specified.

Table 4–9. Approximate Mixing Water Requirements for Different Slumps and Maximum Sizes of Aggregates

Slump, in.	Water, gal per cu yd of concrete, for indicated maximum sizes of aggregate							
	⅜ in.	½ in.	¾ in.	1 in.	1½ in.	2 in.	3 in.	6 in.
Non-air-entrained concrete								
1 to 2	42	40	37	36	33	31	29	25
3 to 4	46	44	41	39	36	34	32	28
5 to 6	49	46	43	41	38	36	34	30
Approximate amount of entrapped air in non-air-entrained concrete, percent.	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	37	36	33	31	29	27	25	22
3 to 4	41	39	36	34	32	30	28	24
5 to 6	43	41	38	36	34	32	30	26
Recommended average total air content, percent	8	7	6	5	4.5	4	3.5	3

Note. These quantities of mixing water are for use in computing cement factors for trial batches. They are maximums for reasonably well-shaped angular coarse aggregates graded within limits of accepted specifications. If more water is required than shown, the cement factor, estimated from these quantities, should be increased to maintain desired water-cement ratio, except as otherwise indicated by laboratory tests for strength. If less water is required than shown, the cement factor, estimated from these quantities, should not be decreased except as indicated by laboratory tests for strength.

4–19. Book Method of Mix Proportioning

The selection of the quantities of cement, water, and fine and coarse aggregates to produce concrete of desired characteristics may be made from tables that have been established for the purpose. These tables have been worked out for aggregates having properties within the limits specified, and will produce concrete of satisfactory quality for such aggregates. The mix proportions so determined are not always the most economical, however, and the trial batch method should be used to design the mix when time is available and the size of the job is large enough to warrant the laboratory effort required. The mix proportions given by the book method may be modified if the desired workability is not obtained. Such modification, if needed, is made by changing the proportions of fine to coarse aggregate or by changing the amount per cubic yard of cement-water paste. (*Do not change the water-cement ratio in making these modifications*). Details of selecting mix characteristics and quantities of materials are given in the following three paragraphs.

a. Select water-cement ratio to meet strength and exposure conditions. Use the smaller value given by table 4–6 and table 4–7. Concrete exposed to the weather in temperate climate or colder zones should contain 4 to 6 percent of entrained air. Determine maximum size of aggregate from dimensions of members or the limitations of available materials. For economy, use the largest possible size. Determine by sieving or estimate the fineness modulus of the sand. Establish the required slump to meet job placement conditions.

b. Tables 4–10*a* and *b* give suggested mixes for air-entrained and non-air-entrained concrete, respectively. Using the water-cement ratio and aggregate size required, select weights of water and cement per cubic yard of concrete. Select weights of fine and coarse aggregates from the columns in the table that are appropriate to the fineness of the sand. If the slump is to be different from the 3 to 4 inch range of table 4–10 refer to the footnote for corrections to the book quantities. For conversion factors, one bag of cement weighs 94 pounds; there are 8.33 pounds of water in 1 gallon.

c. Optimum aggregate gradation depends upon the proportion of fine to coarse aggregate. The volume of coarse aggregate per cubic yard of concrete is given in table 4-11 for four values of fineness modulus of fine aggregate. This table may be used to estimate mix quantities when the range in table 4-10 is exceeded. Determine water-cement ratio (tables 4-6 and 4-7), water content (tables 4-8 and 4-9), and percent of entrained air. Find the volume of coarse aggregate per cubic yard from table 4-11. Calculate absolute volumes in cubic feet, of water, cement, air, and coarse aggregate per cubic yard. Absolute volume in cubic feet is the weight of material divided by 62.4 times specific gravity. For aggregate, use saturated surface dry specific gravity. Subtract this total from 27 cubic feet to get the absolute volume of fine aggregate and convert this to weight.

a. Air Entrained Concrete

Water-cement ratio Gal per sack	Maximum size of aggregate inches	Air content (entrapped air) percent	Water gal per cu yd of concrete	Cement sacks per cu yd of concrete	With fine sand - fineness modulus = 2.50		
					Fine aggregate per cent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
4.5	3/8	7.5	41	9.1	50	1250	1260
	1/2	7.5	39	8.7	41	1060	1520
	3/4	6	36	8.0	35	970	1800
	1	6	34	7.8	32	900	1940
	1 1/2	5	32	7.1	29	870	2110
5.0	3/8	7.5	41	8.2	51	1330	1260
	1/2	7.5	39	7.8	43	1140	1520
	3/4	6	36	7.2	37	1040	1800
	1	6	34	6.8	33	970	1940
	1 1/2	5	32	6.4	31	930	2110
5.5	3/8	7.5	41	7.5	52	1390	1260
	1/2	7.5	39	7.1	44	1190	1520
	3/4	6	36	6.5	38	1090	1800
	1	6	34	6.2	34	1010	1940
	1 1/2	5	32	5.8	32	970	2110
6.0	3/8	7.5	41	6.8	53	1430	1260
	1/2	7.5	39	6.5	45	1230	1520
	3/4	6	36	6.0	38	1120	1800
	1	6	34	5.7	35	1040	1940
	1 1/2	5	32	5.3	32	1010	2110
6.5	3/8	7.5	41	6.3	54	1460	1260
	1/2	7.5	39	6.0	45	1260	1520
	3/4	6	36	5.5	39	1150	1800
	1	6	34	5.2	36	1080	1940
	1 1/2	5	32	4.9	33	1040	2110
7.0	3/8	7.5	41	5.9	54	1500	1260
	1/2	7.5	39	5.6	46	1300	1520
	3/4	6	36	5.1	40	1180	1800
	1	6	34	4.9	36	1100	1940
	1 1/2	5	32	4.6	33	1060	2110
7.5	3/8	7.5	41	5.5	55	1530	1260
	1/2	7.5	39	5.2	47	1330	1520
	3/4	6	36	4.8	40	1210	1800
	1	6	34	4.5	37	1140	1940
	1 1/2	5	32	4.3	34	1090	2110
8.0	3/8	7.5	41	5.1	55	1560	1260
	1/2	7.5	39	4.9	47	1360	1520
	3/4	6	36	4.5	41	1240	1800
	1	6	34	4.3	37	1160	1940
	1 1/2	5	32	4.0	34	1110	2110

Note. Increase or decrease water per cubic yard by 3 percent for each increase or decrease of 1 in. in slump, then calculate quantities by absolute volume method. For manufactured fine aggregate, increase percentage of fine aggregate by 3 and water by 17 lb per cubic yard of concrete. For less workable concrete, as in pavements, decrease percentage of fine aggregate by 3 and water by 8 lb per cubic yard of concrete.

Table 4-10. Trial Mixes for Concrete of Medium Consistency – Continued
a. Air-Entrained Concrete – Continued

With average sand - fineness modulus = 2.75			With coarse sand - fineness modulus = 2.90		
Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete	Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
53	1320	1190	54	1360	1150
44	1130	1450	46	1180	1400
38	1040	1730	39	1090	1680
34	970	1870	36	1010	1830
32	950	2030	33	990	1990
54	1400	1190	56	1440	1150
46	1210	1450	47	1260	1400
39	1110	1730	41	1160	1630
36	1040	1870	37	1080	1830
33	1010	2030	35	1050	1990
55	1460	1190	57	1500	1150
46	1260	1450	48	1310	1400
40	1160	1730	42	1210	1680
37	1080	1870	38	1120	1830
34	1050	2030	35	1090	1990
56	1500	1190	57	1540	1150
47	1300	1450	49	1350	1400
41	1190	1730	42	1240	1680
37	1110	1870	39	1150	1830
35	1090	2030	36	1130	1990
56	1530	1190	58	1570	1150
48	1330	1450	50	1380	1400
41	1220	1730	43	1270	1680
38	1150	1870	39	1190	1830
36	1120	2030	37	1160	1990
57	1570	1190	58	1610	1150
49	1370	1450	50	1420	1400
42	1250	1730	44	1300	1680
38	1170	1870	40	1210	1830
36	1140	2030	37	1180	1990
57	1600	1190	59	1640	1150
49	1400	1450	51	1450	1400
43	1280	1730	44	1330	1680
39	1210	1870	41	1250	1830
37	1170	2030	38	1210	1990
58	1630	1190	59	1670	1150
50	1430	1450	51	1480	1400
43	1310	1730	44	1360	1680
40	1230	1870	41	1270	1830
37	1190	2030	38	1230	1990

					With fine [illegible] fineness modul[illegible] 2.50		
Water-cement ratio Gal per sack	Maximum size of aggregate inches	Air content (entrapped air) percent	Water gal per cu yd of concrete	Cement sacks per cu yd of concrete	Fine aggregate per cent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu.yd of concrete
4.5	3/8	3	46	10.3	50	1240	1260
	1/2	2.5	44	9.8	42	1100	1520
	3/4	2	41	9.1	35	960	1800
	1	1.5	39	8.7	32	910	1940
	1 1/2	1	36	8.0	29	880	2110
5.0	3/8	3	46	9.2	51	1330	1260
	1/2	2.5	44	8.8	44	1180	1520
	3/4	2	41	8.2	37	1040	1800
	1	1.5	39	7.8	34	990	1940
	1 1/2	1	36	7.2	31	960	2110
5.5	3/8	3	46	8.4	52	1390	1260
	1/2	2.5	44	8.0	45	1240	1520
	3/4	2	41	7.5	38	1090	1800
	1	1.5	39	7.1	35	1040	1940
	1 1/2	1	36	6.5	32	1000	2110
6.0	3/8	3	46	7.7	53	1440	1260
	1/2	2.5	44	7.3	46	1290	1520
	3/4	2	41	6.8	39	1130	1800
	1	1.5	39	6.5	36	1080	1940
	1 1/2	1	36	6.0	33	1040	2110
6.5	3/8	3	46	7.1	54	1480	1260
	1/2	2.5	44	6.8	46	1320	1520
	3/4	2	41	6.3	39	1190	1800
	1	1.5	39	6.0	37	1120	1940
	1 1/2	1	36	5.5	34	1070	2110
7.0	3/8	3	46	6.6	55	1520	1260
	1/2	2.5	44	6.3	47	1360	1520
	3/4	2	41	5.9	40	1200	1800
	1	1.5	39	5.6	37	1150	1940
	1 1/2	1	36	5.1	34	1100	2110
7.5	3/8	3	46	6.1	55	1560	1260
	1/2	2.5	44	5.9	48	1400	1520
	3/4	2	41	5.5	41	1240	1800
	1	1.5	39	5.2	38	1190	1940
	1 1/2	1	36	4.8	35	1130	2110
8.0	3/8	3	46	5.7	56	1600	1260
	1/2	2.5	44	5.5	48	1440	1520
	3/4	2	41	5.1	42	1280	1800
	1	1.5	39	4.9	39	1220	1940
	1 1/2	1	36	4.5	35	1160	2110

Note. Increase or decrease water per cubic yard by 3 percent for each increase or decrease of 1 in. in slump, then calculate quantities by absolute volume method. For manufactured fine aggregate, increase percentage of fine aggregate by 3 and water by 17 lb per cubic yard of concrete. For less workable concrete, as in pavements, decrease percentage of fine aggregate by 3 and water by 8 lb per cubic yard of concrete.

Table 4-10. Trial Mixes for Concrete of Medium Consistency — Continued
b. Non-Air-Entrained Concrete — Continued

With average sand - fineness modulus = 2.75			With coarse sand - fineness modulus = 2.90		
Fine aggregate percent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete	Fine aggregate per cent of total aggregate	Fine aggregate lb per cu yd of concrete	Coarse aggregate lb per cu yd of concrete
52	1310	1190	54	1350	1150
45	1170	1450	47	1220	1400
37	1030	1730	39	1080	1680
34	980	1870	36	1020	1830
32	960	2030	33	1000	1990
54	1400	1190	56	1440	1150
46	1250	1450	48	1300	1400
39	1110	1730	41	1160	1680
36	1060	1870	38	1100	1830
34	1040	2030	35	1080	1990
55	1460	1190	57	1500	1150
47	1310	1450	49	1360	1400
40	1160	1730	42	1210	1680
37	1110	1870	39	1150	1830
35	1080	2030	36	1120	1990
56	1510	1190	57	1550	1150
48	1360	1450	50	1410	1400
41	1200	1730	43	1250	1600
38	1150	1870	39	1190	1830
36	1120	2030	37	1160	1990
57	1550	1190	58	1590	1150
49	1390	1450	51	1440	1400
42	1240	1730	43	1290	1680
39	1190	1870	40	1230	1830
36	1150	2030	37	1190	1990
57	1590	1190	59	1630	1150
50	1430	1450	51	1480	1400
42	1270	1730	44	1320	1680
39	1220	1870	41	1260	1830
37	1180	2030	38	1220	1990
58	1630	1190	59	1670	1150
50	1470	1450	52	1520	1400
43	1310	1730	45	1370	1600
40	1260	1870	42	1300	1830
37	1210	2030	39	1250	1990
58	1670	1190	60	1710	1150
51	1520	1450	53	1560	1400
44	1350	1730	45	1400	1680
41	1290	1870	42	1330	1830
38	1250	2030	39	1280	1990

Maximum size of aggregate, in.	Fineness modulus of fine aggregate			
	2.40	2.60	2.80	3.00
	Coarse aggregate, cu ft per cu yd*			
⅜	13.5	13.0	12.4	11.9
½	15.9	15.4	14.8	14.3
¾	17.8	17.3	16.7	16.2
1	19.2	18.6	18.1	17.6
1½	20.2	19.7	19.2	18.6
2	21.1	20.5	20.0	19.4
3	22.1	21.6	21.1	20.5

*Volumes are based on aggregates in dry-rodded condition. These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction, they may be increased about 10 percent. When placement is to be by pump, they should be decreased about 10 percent.

4–20. Trial Batch Method of Mix Proportioning

The trial batch method is a simple field method which incorporates many of the best features of several other methods, and it may be used under almost any circumstances. The trial batch method is capable of producing a satisfactory mix incorporating the materials which will be used in the project with a minimum of tests. The procedure consists of 6 steps:

a. The first step is the selection of the water-cement ratio, which must satisfy the requirements of strength, durability, and watertightness. Tables 4–6 and 4–7 list suggested W/C ratios. Since the W/C ratio is the most important factor, it must be held constant and other variables used to control the other requirements. It is extremely important to remember that a change in the W/C ratio will alter the characteristics of the hardened concrete.

b. The second step is to determine the workability or consistency desired. Consistency is measured by the slump test (para 4–23) and is expressed in inches of slump. Slump (in inches) is inversely proportional to the stiffness of the mix. See table 4–8 for recommended values of slump for different types of construction. In the absence of any other criteria, concrete should be placed as "stiff" as possible, yet maintaining a homogeneous voidless mass.

c. The third step is the calculation of weights of cement, water, or fine and coarse aggregates required for the trial batch. This design of a small batch mix, usually based on one-fifth (18.8 pounds) or one-tenth (9.4 pounds) of a bag of cement, follows the procedure outlined in para 4–19. This step may be omitted if data needed for this calculation are not available, and time does not permit their determination.

d. The fourth step is the mixing of a trial batch. A convenient batch quantity must be determined. Although, on large projects, a batch using the full tity batch is more convenient. More reliable results will be obtained from larger batches, particularly for larger size coarse aggregate. The batch must be large enough for machine mixing if the concrete is air-entraining. All quantities of materials used are measured and recorded. The slump, air content (para 4–23), and observations of workability are recorded.

e. The fifth step is to determine the amount of concrete which will result from a one-bag batch. Since the proportions for a one-fifth bag batch have been determined, multiplying the weights by five will give the weight of each ingredient in a one-bag batch. Converting these weights to absolute volumes and adding the volumes will give the volume of concrete obtained from a one-bag batch.

f. The sixth step is the calculation of the cement factor (CF). This is a numerical value by which the amount of each ingredient (in a one-bag batch) is multiplied and results in the amount of each ingredient needed to fill or batch a mixer. The cement factor is obtained by dividing the volume of the job (or mixer) by the yield per bag of cement.

For example, if a mixture has a 16-cubic foot capacity (no overload) and a one-bag batch yields 4 cubic feet of concrete, the (CF) equals 16/4 or 4. The amount of each ingredient in the one-bag batch is multiplied by four and this is the quantity which must be placed in the mixer for a full load.

4–21. Preparation for Trial Batch Procedure

a. Apparatus. The test procedures which are described require the following equipment:

Bucket, 14 quart
Slump-cone molds
Electric oven
Concrete-mixing pan, 24 x 31 x 6 inch
Steel tamping rod, ⅜-inch diameter by 24-inch long
Steel rule
Scales, 300 pound capacity
Kitchen scoop
Square point shovel
Brick trowels
Wood float
Graduates, 25 ml

Some of this equipment is shown in figure 4–4.

b. Preparation of Aggregates. Some of the physical characteristics of the aggregates to be used must be determined before the mix design can be carried out. Tests will be conducted to provide the following necessary information:

(1) Gradation of both fine and coarse aggregates (para 4–9).

(2) Bulk specific gravities (saturated, surface-dry condition) and absorption of both fine and coarse aggregates (para 4–10).

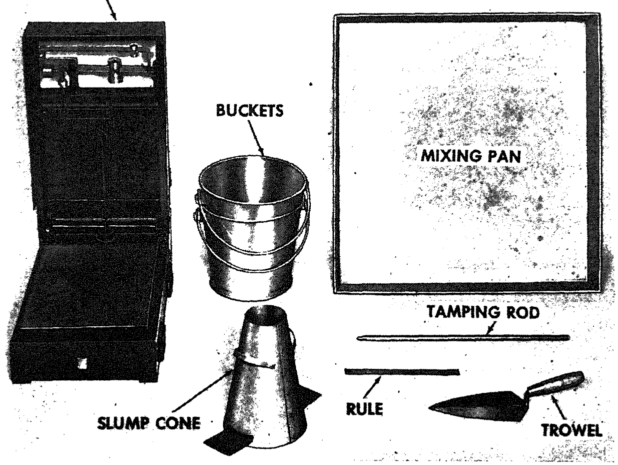


Figure 4–4. Apparatus for trial batch method of mix proportioning of portland cement concrete.

(3) Surface moisture of both fine and coarse aggregates (para 4–10*e* and 4–10*f*).

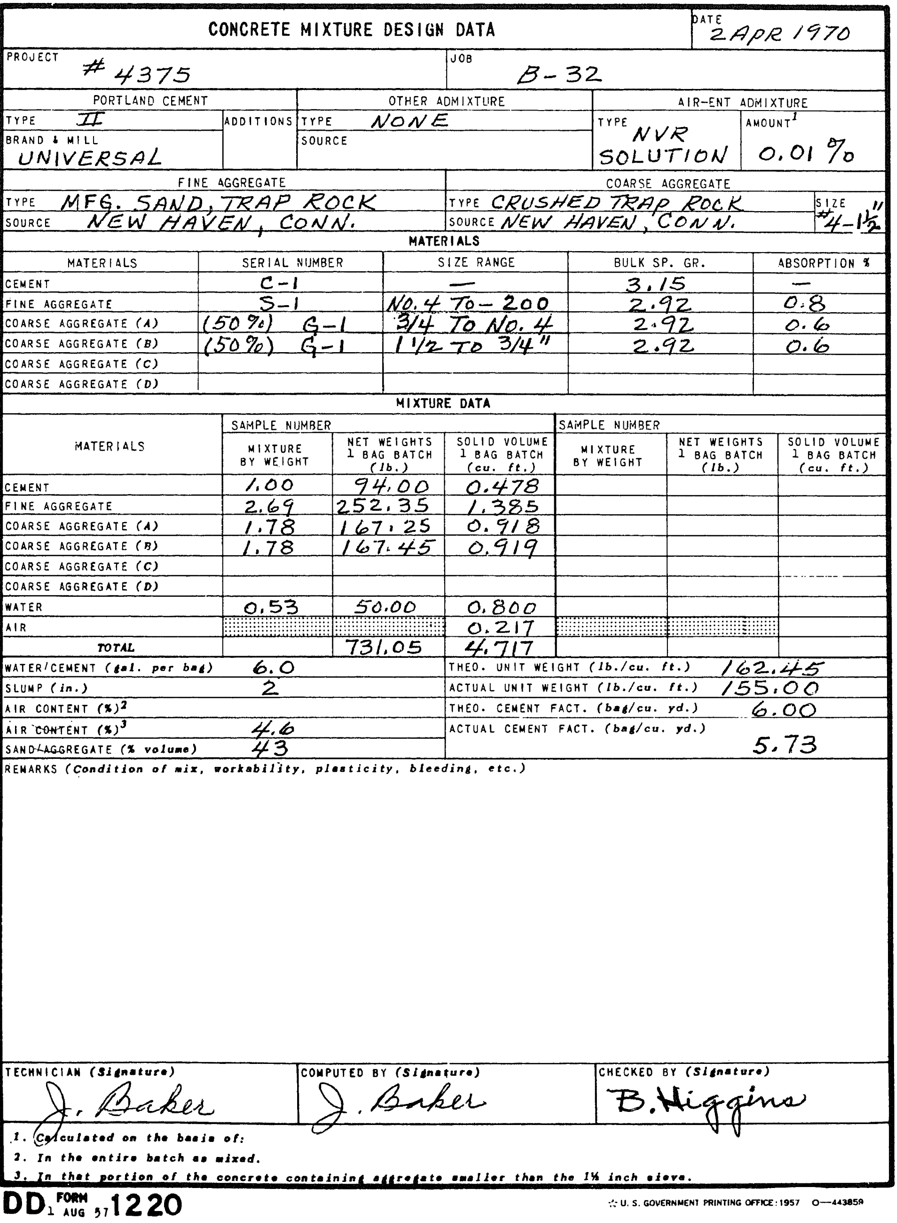
(4) If time permits, soak the aggregates for several hours prior to use, allow them to dry to a saturated, surface-dry condition, and place them in covered containers to keep in this condition until use. Corrections for surface moisture then will not be required.

(5) If the course aggregate to be used contains particles larger than ¾ inch, it should be separated into two or more fractions. *For example*, it may be divided into the following sizes: No. 4 to ¾ inch, ¾ inch to 1½ inches, 1½ to 3 inches. The two or more fractions are then recombined in the proportions necessary to produce the best possible gradation. Desirable coarse aggregate gradings are given in table 4–2. *Example*. Assume that the coarse aggregate to be used contains material from the No. 4 sieve to 1½ inches in size. It should be divided into two size ranges: No. 4 to ¾ inch, and ¾ to 1½ inches. The two fractions are then recombined in such proportions as to most nearly give the grading called for in table 4–2. This proportioning procedure and the necessary computations are given in table 4–12.

(6) The result of the preliminary tests are entered in the appropriate spaces on DD Form 1220 (Concrete Mixture Design Data), fig. 4–5.

Table 4–12. *Example of Calculations, Aggregate Blending*

Sieve size	Separate sizes individual % retained		50%	50%	Combined grading 50-50			Desired grading	
	No. 4 to ¾ in.	¾ to 1½ in.	No. 4 to ¾ in.	¾ to 1½ in.	Individual % ret.	Cumulative %		Individual % ret.	Cumulative % Pass.
						Ret.	Pass.		
1½ in.							100. 0	0. 0	100. 0
1 in.		60. 0		30. 0	30. 0	30. 0	70. 0	28. 4	71. 6
¾ in.		36. 0		18. 0	18. 0	48. 0	52. 0	16. 9	54. 7
½ in.	34. 0	4. 0	17. 0	2. 0	19. 0	67. 0	33. 0	20. 1	34. 6
⅜ in.	24. 0		12. 0		12. 0	79. 0	21. 0	13. 2	21. 4
No. 4	40. 0		20. 0		20. 0	99. 0	1. 0	21. 4	0. 0
Pan	2. 0		1. 0		1. 0	100. 0			
Total	100. 0	100. 0	50. 0	50. 0	100. 0			100. 0	



CONCRETE MIXTURE DESIGN DATA

DATE 2 APR 1970

PROJECT # 4375

JOB B-32

PORTLAND CEMENT		OTHER ADMIXTURE	AIR-ENT ADMIXTURE	
TYPE II	ADDITIONS	TYPE NONE	TYPE NVR SOLUTION	AMOUNT[1] 0.01 %
BRAND & MILL UNIVERSAL		SOURCE		

FINE AGGREGATE	COARSE AGGREGATE	
TYPE MFG. SAND, TRAP ROCK	TYPE CRUSHED TRAP ROCK	SIZE #4-1½"
SOURCE NEW HAVEN, CONN.	SOURCE NEW HAVEN, CONN.	

MATERIALS

MATERIALS	SERIAL NUMBER	SIZE RANGE	BULK SP. GR.	ABSORPTION %
CEMENT	C-1	—	3.15	—
FINE AGGREGATE	S-1	NO. 4 TO - 200	2.92	0.8
COARSE AGGREGATE (A)	(50%) G-1	3/4 TO NO. 4	2.92	0.6
COARSE AGGREGATE (B)	(50%) G-1	1 1/2 TO 3/4"	2.92	0.6
COARSE AGGREGATE (C)				
COARSE AGGREGATE (D)				

MIXTURE DATA

MATERIALS	SAMPLE NUMBER			SAMPLE NUMBER		
	MIXTURE BY WEIGHT	NET WEIGHTS 1 BAG BATCH (lb.)	SOLID VOLUME 1 BAG BATCH (cu. ft.)	MIXTURE BY WEIGHT	NET WEIGHTS 1 BAG BATCH (lb.)	SOLID VOLUME 1 BAG BATCH (cu. ft.)
CEMENT	1.00	94.00	0.478			
FINE AGGREGATE	2.69	252.35	1.385			
COARSE AGGREGATE (A)	1.78	167.25	0.918			
COARSE AGGREGATE (B)	1.78	167.45	0.919			
COARSE AGGREGATE (C)						
COARSE AGGREGATE (D)						
WATER	0.53	50.00	0.800			
AIR			0.217			
TOTAL		731.05	4.717			

WATER/CEMENT (*gal. per bag*)	6.0	THEO. UNIT WEIGHT (*lb./cu. ft.*)	162.45
SLUMP (*in.*)	2	ACTUAL UNIT WEIGHT (*lb./cu. ft.*)	155.00
AIR CONTENT (%)[2]		THEO. CEMENT FACT. (*bag/cu. yd.*)	6.00
AIR CONTENT (%)[3]	4.6	ACTUAL CEMENT FACT. (*bag/cu. yd.*)	5.73
SAND/AGGREGATE (% *volume*)	43		

REMARKS (*Condition of mix, workability, plasticity, bleeding, etc.*)

TECHNICIAN (*Signature*)	COMPUTED BY (*Signature*)	CHECKED BY (*Signature*)
J. Baker	J. Baker	B. Higgins

1. Calculated on the basis of:

2. In the entire batch as mixed.

3. In that portion of the concrete containing aggregate smaller than the 1½ inch sieve.

DD FORM 1 AUG 57 1220

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Figure 4-5. Data sheet, concrete mix design.

Entraining Admixture. If an air-entraining
to be used in the concrete, the proper amount
material is obtained and placed in a suitable
er. The amount of admixture required varies
rably, depending upon the type used. If neu-
vinsol resin is used, approximately 0.01 per-
weight of the cement will be required. This
illiliters for a one-bag batch.

xing Pan. Before the trial batch can be mixed,
de of the mixing pan must be coated with
similar to that which will be contained in the
tch. After the materials for the trial batch
en prepared, the same procedure is followed
reparation of a one-twentieth-bag batch. All
es will be one-quarter of those used in prepa-
f the one-fifth-bag trial batch. The one-
h-bag batch is placed in the mixing pan and
ccording to the procedure described in para-
–22. This batch is discarded after making
t the inside of the pan is well coated with
If the trial batch is not mixed immediately
5 minutes), the pan is covered with wet bur-
eep the mortar from drying out.

rial Batch Procedure

l batch procedures are as follows:
a one-tenth bag trial batch if the aggregate
not exceed ¾ inch and if hand mixing is
r larger aggregate, and for machine mixing
ncrete is air-entraining, use a one-fifth bag
trial mix. Weigh out the required amount
t, 9.4 pounds or 18.8 pounds, respectively.
m the specified water-cement ratio, the
f water to be used is calculated and weighed
weight of mixing water required may be
d by the use of one of two formulas.
The first equation is used when the water-
atio (W/C) is given as a decimal fraction
t, and the second when the water-cement
expressed in gallons of water per bag of

$$\text{eight of water, pounds} = (W/C)\ 9.4,$$

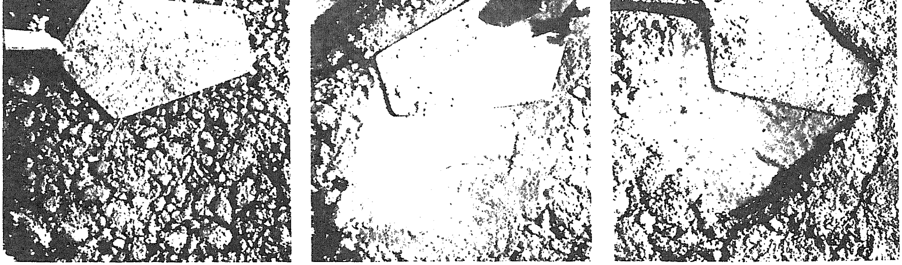
$$\text{eight of water, pounds} = \frac{(W/C)\ 8.33}{10.0}$$

The amount of mixing water calculated
ased upon the assumption that the aggre-
in a saturated, surface-dry condition. If the
s are not in this condition, the amount of
d must be corrected. If free moisture (above the moisture content of the aggregate in a saturated, surface-dry condition) is present in the aggregate, the amount of free moisture must be deducted from the calculated weight of mixing water. If the moisture content of the aggregate is less than that which corresponds to a saturated, surface-dry condition, enough water must be added to the calculated weight of mixing water to bring the aggregate to a saturated, surface-dry condition.

c. If an air-entraining admixture is to be used, calculate the required amount and measure it carefully. Take about ¾ of the mixing water, add the air-entraining agent to it, and mix thoroughly.

d. Weigh out enough fine and coarse aggregate to exceed the requirements of the trial batch (about 35 pounds of fine aggregate and 55 pounds of coarse aggregate for a one-tenth bag batch), place in separate containers and cover with damp cloths to prevent change in water content.

e. About 30 pounds of the coarse aggregate and about 20 pounds of the fine aggregate are placed in the mixing pan and thoroughly mixed with the shovel. All of the cement is added and mixed with the aggregates. After the cement and aggregates have been mixed, a flat-topped pile is formed in the center of the pan. The center of the pile is hollowed out to form a shallow crater. The portion of the mixing water containing the air-entraining agent is poured into the crater in the pile of cement and aggregate. The aggregates and cement are mixed with the water by shoveling them into the crater. Splashing water into the pan outside the crater should be avoided. Carefully add the remainder of the mixing water together with more aggregates as needed to prevent loss of mixing water. Additional coarse and fine aggregate (in about a ratio of 2: 1) may be added as needed to reduce the consistency to the desired slump. Care should be taken to prevent harshness due to excessive amounts of coarse aggregate, or oversanding. The concrete should be troweled occasionally in order to estimate the texture of the mixture. If the mixture appears too smooth during troweling (too easily finished), more coarse aggregate is required. If considerable effort in troweling is necessary to bring about a smooth finish, the mixture is too harsh and more sand is required. Properly proportioned and mixed concrete is cohesive and appears rough around the edges, though with a medium amount of troweling it will present a smooth, dense surface. The appearance of a properly proportioned mix is shown in figure 4–6.



A—Undersanded B—Correct proportions C—Oversanded

Figure 4–6. Appearance of concrete mixtures.

f. When the trial mix appears to have the desired consistency, perform the slump test, measure the air content (if air-entrained mix), and take notes on workability. Revise the mix by adding aggregates, as needed, until satisfactory proportions are obtained. Weigh the remaining aggregates and subtract from the original weight to obtain the amounts used for each aggregate. Record all weights of materials on trial mix data form.

g. Usually a single trial batch does not supply enough information, since it is almost certain to be either oversanded or too harsh, or to miss the desired slump or air content. After calculating the results of the first batch, additional batches should be made using different weights of material obtained by adjusting the quantities used in the initial batch. Each batch is mixed according to the procedure previously described. Adjustments in the mix are made until the best possible mixture is obtained.

4—23. Tests of Fresh Concrete

After the concrete is first mixed several tests are used to determine its quality such as—

a. Slump Test. When the mixture apears to have reached the desired consistency, a slump test should be made. The slump cone (a metal mold, fig. 4–7) is dampened with a moist cloth and placed on a flat, smooth, nonabsorbent surface. The slump cone is filled in three aproximately equal volumes, using a representative sample of the concrete. In placing each scoopful of concrete in the slump cone, the scoop is moved around to insure equal distribution of concrete in the cone. Each of the three layers of concrete is rodded with 25 strokes of the tamping rod. The strokes are distributed in a uniform manner over the cross section of the slump cone. Each stroke should penetrate into (not through) the underlying layer, with the bottom layer being rodded throughout its entire depth. After the top layer of concrete has been rodded, the surface of the concrete is struck off with a trowel so that the slump cone is exactly filled. The slump cone is removed from the concrete by raising it carefully in a vertical direction. The slump is measured immediately by determining the difference between the height of the slump cone and the height of the pile of concrete at its vertical axis, as illustrated in figure 4–7. The results are recorded on DD Form 1220 (fig. 4–5), in inches to the nearest 1/4 inch. The desired slump will be established by the officer in charge. (See table 4–8 for typical ranges in slump.) Reference may be made to TM 5-742 for additional information, if desired. After the slump measurement is complete, the side of the pile of concrete on which the slump was determined should be tapped gently with the tamping rod. The behavior of the concrete under this treatment is a valuable indication of the cohesiveness, workability, and placeability of the mixture. A well-proportioned, workable mix will gradually slump to lower elevations and retain its original identity, while a poor mix will crumble, segregate, and fall apart.

b. Air Content Testing. An air-entraining admixture is added to the concrete mix so that enough air will be entrained to improve workability and durability of the mixture, but not enough to substantially reduce the strength. Air-entraining cements may also be available for use in some military situations. The desired amount of air is generally from 3 to 6 percent of the total mix.

(1) *Apparatus.* The equipment for determining the percentage of entrained air is included in the boxed test kit. The basic tool is the pressure-type indicator. The equipment furnished in these kits varies with manufacturers. Each kit contains complete equipment for conducting this test, including a detailed instruction pamphlet and the calibration procedure for the particular meter. Typical test apparatus is shown in figure 4–8.

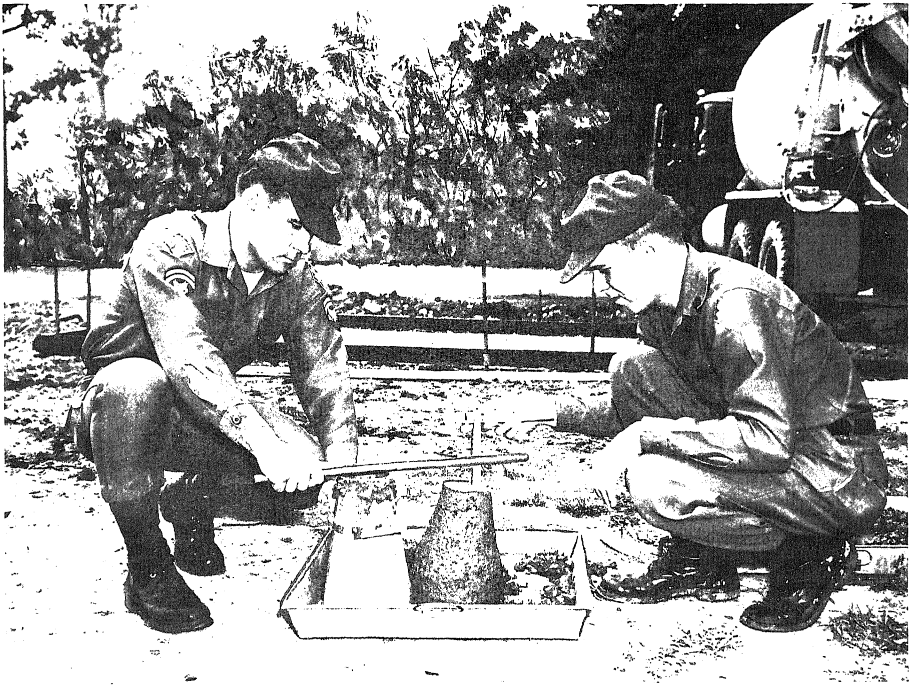


Figure 4-7. Measuring slump of fresh concrete.

(2) *Calibration of apparatus.* Before the air content of a concrete mixture can be determined, it is necessary that the entrained air indicator be calibrated accurately and that the correction factor for the aggregate contained in the concrete be determined.

(3) *Testing procedure.* The following is the step-by-step test procedure in determining the air content:

(*a*) The measuring bowl of the apparatus is filled with three equal layers of the concrete to be tested. Each layer is consolidated by 25 blows of the rodding tool evenly distributed over the surface of the layer. After rodding each layer, the sides of the bowl are tapped smartly 10 to 15 times with the mallet until the holes left by the rodding are leveled out, and no large bubbles of air appear on the surface of the layer. While rodding the first layer, care should be taken not to strike the bottom of the measuring bowl too hard. The second and third layers should be rodded with only enough force for the rod to penetrate the surface of the previous layer. The bowl should be overfilled by the third layer, and after rodding, the excess concrete should be removed by sliding the strike-off bar across the top flange with a sawing motion until the concrete is just level with the top of the bowl.

(*b*) The remainder of the test will depend on the type of meter furnished. The detailed procedure is outlined in the instruction book furnished with each meter.

(*c*) The calculations for determining the entrained air content are given in the instruction book furnished with the air meter.

c. Unit Weight. A cylindrical metal measure of either $1/10$-, $1/5$-, or $1/2$-cubic foot capacity is filled with the fresh concrete and rodded in three layers as described for the slump test. After each layer is rodded, the sides of the container are tapped several times with the rod to remove any air pockets. The containers are rodded 25 times per layer. After filling and consolidating, the top surface is struck off with a heavy, flat metal cover plate to insure that the measure is level full. Clean all excess concrete from the surface of the container and weigh. Measure the tare weight of the clean, dry container. The measure

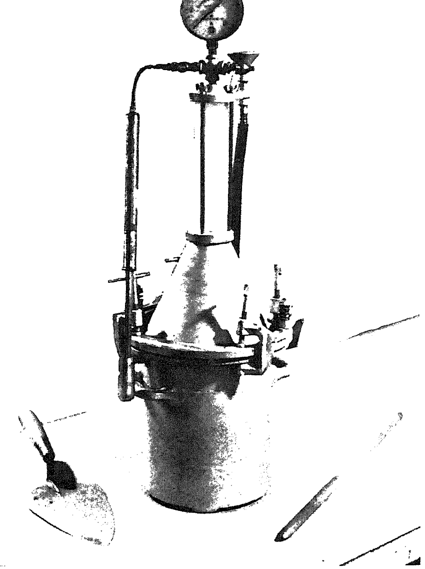


Figure 4–8. Apparatus, air content test.

is calibrated if necessary to determine its volume by accurately filling it with water at 62° F. (16.7° C.), weighing and dividing the weight of water by 62.4 pounds per cubic foot to get the volume of container.

4–24. Calculations

a. The volume of concrete which is produced per sack of cement is called the yield. Yield for concrete made without an air-entraining agent is computed by assuming that the ingredients form a solid mass with

is desirable that the yield be as high as possible, consistent with the other desired properties of the concrete mixture. The yield is also useful in estimating the quantity of each material required to produce a given amount of concrete.

b. Correct the actual weights used in the trial mix for free surface moisture if any is present in the aggregates. Record corrected values (SSD weights). The weights of materials used are multiplied by 94 pounds and divided by the weight of cement used to get the weights for a one-bag batch. Calculate the yield for a one-bag batch by dividing the total weight of a material by the unit weight of concrete. The cement factor is the number of 94-pound bags of cement required for a cubic yard of concrete. It is calculated by dividing the yield per bag into 27 (cubic feet per cubic yard). Then weights of all materials needed for a cubic yard are found by multiplying the weights for a one-bag batch by the cement factor. Convert pounds of water to gallons per cubic yard by dividing the weight of water by 8.33 pounds per gallon. The fine aggregate expressed in percent by weight of total aggregate is found as:

$$\frac{\text{weight of fine aggregate}}{\text{weight of fine aggregate plus weight of coarse aggregate}}\ (100)$$

c. The cement factor will vary with the water-cement ratio and the maximum size and particle shape of the coarse aggregate used. The cement factor might be as high as 9.0 bags per cubic yard with pea-gravel concrete and as low as 5.0 or 5.5 bags per cubic yard with angular, 1½-inch stone, both having a water-cement ratio of 6.0 gallons per bag. Cement factors for structural and paving concrete containing 1½-inch aggregate and a water cement ratio of 5.5 gallons per bag might vary between 5.5 and 7.0 bags of cement per cubic yard.

Section V. FLEXURAL STRENGTH (MODULUS OF RUPTURE) TEST

4–25. Introduction

The purpose of the test described in this section is to determine the flexural strength of concrete. A determination of the flexural strength is frequently necessary as a part of the design of concrete mixtures to check compliance with established specification or even to provide information necessary to the design of an engineering structure. Beam forms for casting test beams from fresh concrete are available in the concrete test set. Although equipment for obtaining sawed specimens is not provided, the test may be performed upon beams sawed from existing concrete structures for evaluation purposes. The equipment furnished includes a tester designed to obtain an index of the flexural strength. Equipment which is necessary for the performance of this test, aside from the loading frame, is as follows:

Concrete beam tester, third-point loading
Wire brush
Wood float
Collapsible mold, for 6- x 6- x 21-inch beams
Mixing pan, 24 x 24 x 3 inches
Proving ring, complete with gage and ring clamps, 10,000-pound capacity
Tamping rod
Square point shovel
Steel straightedge
Brick trowel

The flexural strength testing assembly is shown in figure 4–9. An alternate testing assembly can be made up from the loading frame and certain of the attachments provided with the CBR test set, the breaker (third-point loading), and the 10,000-pound-capacity proving ring. This assembly is shown in figure 4–9 (alternate testing assembly).

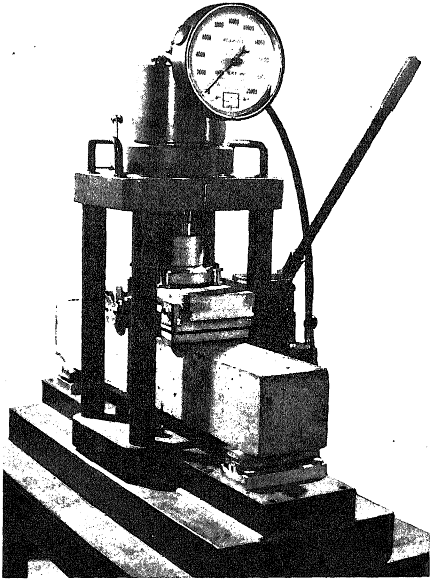


Figure 4–9. Flexural strength testing assembly for concrete beam—Standard 6- x 6- x 21-inch beam ready for testing.

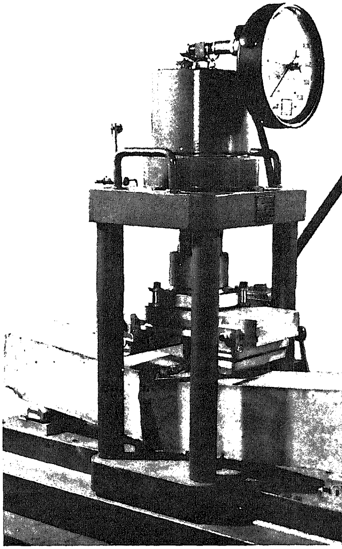


Figure 4–9. Beam broken by third-point loading

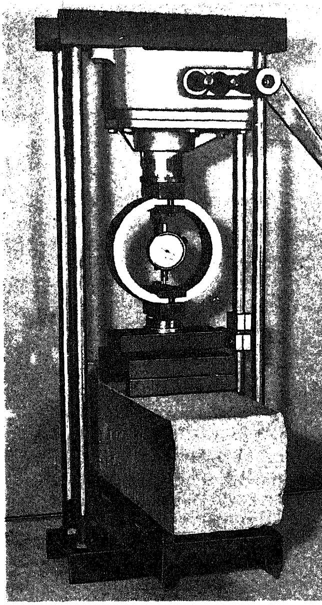


Figure 4–9. Alternate testing assembly using Ol ment—Continued

4–26. Test Beams

The concrete from which the test beams are to be made is mixed in accordance with the procedure described in paragraph 4–22, except that three one-fifths-bag batches are required. This will produce approximately 3 cubic feet of concrete. After the concrete has been mixed, the slump tests is conducted in accordance with the procedure described in paragraph 4–23. If air-entrained concrete is being made, the air content is determined by the procedure described in paragraph 4–23. If the slump and air content are within the desired limits, the concrete used to conduct these tests is returned to the mixing pan. The entire batch is remixed just enough to produce a homogeneous mass. Flexure test beams are used to check the quality of concrete being produced in the field, particularly in constructing pavements. When the concrete has cured to produce an adequate beam strength, traffic may use the pavement. Concrete is sampled from the production batches as required and beams are molded, cured, and tested as described in the following procedures.

a. Forming Beams. A concrete-beam mold is assembled and the inside of the mold lightly oiled. The mold is filled with two layers of concrete, each approximately 3 inches in depth. Each layer is consolidated by hand rodding, using 40 strokes evenly distributed over the surface of the layer. A light spading of the concrete along the sides of the mold with a trowel helps in removal of surface voids. The top surface is struck off with a straightedge and finished with a wood float.

b. Number of Specimens. Five specimens are made for each mix design being evaluated. Two of the beams are broken at the age of 7 days. This will permit an early evaluation of the mix design. The remaining three beams are broken at the age of 28 days. Specimens containing standard portland cement generally will develop about 80 percent of their 28-day strength during the first 7 days of curing.

c. Curing. A suitable identifying label is placed on the finished surface of the specimen and the entire specimen, still in the mold, covered with a double thickness of wet burlap. About 73° F. (23° C.) is the most satisfactory curing temperature for concrete. Therefore, an effort should be made to store the test specimens at approximately that temperature. The specimens are removed from the molds after 24 hours of curing. The beams are stored under water, in moist sand, sawdust, or earth, or under a double thickness of wet burlap, so that they will be kept continuously wet until they are tested.

4–27. Testing Procedure

The loading device is assembled as shown in figure 4–9. The test beam is turned so that the finished surface is to the side and centered in the loading assembly. The testing apparatus is operated until the loading blocks are brought into contact with the upper surface of the beam. Care should be taken to secure full contact between the loading (and supporting) surfaces and the beam. If the surface of the specimen is so rough that full contact is not secured, the specimen should be ground to secure full contact. Load may be applied at a rate such that the increase in extreme fiber stress in the beam does not exceed 150 pounds per square inch per minute. The extreme fiber stress corresponding to any load may be estimated from the equation given in paragraph 4–28. Readings are obtained on the proving ring dial, and are converted to corresponding total loads in pounds by application of the proving ring constant. Aside from controlling the rate of application of the load, the only reading necessary is that which corresponds to the maximum load applied to the test beam. After the specimen has been broken, the dimensions of the cross section at which failure occurred are obtained to the nearest 0.1 inch. These dimensions are taken to be the average width and average depth at the section of failure.

4–28. Modulus of Rupture Calculations

a. If the specimen broke within the middle third of the span length, the modulus of rupture is calculated as follows:

$$R=\frac{PL}{bd^2}$$

Where:

R=modulus of rupture in pounds per square inch,

P=maximum applied load in pounds,

L=span length in inches,

b=average width of specimen in inches, and

d=average depth of specimen in inches.

b. If the specimen broke outside the middle third of the span length by not more than 5 percent of the span length, the modulus of rupture is calculated as follows:

$$R=\frac{3Pa}{bd^2},$$

where a is the distance in inches between the line of fracture and the nearest support, measured along the center line of the bottom surface of the beam, and other symbols have meanings given in *a* above.

c. If the specimen broke outside of the middle third of the span length by more than 5 percent of the span length, the results of the test will be discarded.

d. The report of the test for flexural strength should include the following:

(1) Identification number.

(2) Average width to the nearest 0.1 inch.

(3) Average depth to the nearest 0.1 inch.
(4) Span length in inches.
(5) Maximum applied load in pounds.
(6) Modulus of rupture calculated to the nearest 5 pounds per square inch.
(7) Defects in specimen.
(8) Age of specimen.

4-29. Conclusions

a. Values of the modulus of rupture vary widely, depending on the concrete tested. Specifications relative to concrete pavements frequently require a modulus of rupture in excess of 600 to 650 pounds per square inch (28-day curing, third-point loading). The flexural strength (modulus of rupture) generally may be expected to be approximately 15 percent of the compressive strength for comparable conditions of age and curing.

b. An approximate relationship between modulus of rupture and compressive strength is

$$f'_c = \frac{R^2}{100},$$

in which f'_c is compressive strength, pounds per square inch, R is modulus of rupture, pounds per square inch.

Section VI. COMPRESSIVE STRENGTH TEST

4-30. Introduction

a. The compressive strength of hardened concrete, as measured by compression tests on standard forms of cylindrical specimens, is used in the design of structures. The standard test specimen is a cylinder 6 inches in diameter by 12 inches long, capped with a suitable material to provide smooth bearing surfaces on each end. Load is applied to the end surfaces through metal platens on the testing machine (cylinder breaker) causing compressive stress in the longitudinal direction of the cylinder.

b. The following equipment is required to mold and test concrete cylinders:

Concrete compression tester, 250,000-pound capacity
Mixing equipment for trial mixes (para 4-21a)
Concrete cylinder molds, 6 by 12 inches
Mixing pan, 24 by 24 by 3 inches
Scoop
Tamping rod
Steel straightedge
Trowel
Square point shovel
Polyethylene sheeting
Burlap
Calipers, 6-inch opening
Steel scale, inches and hundredths

c. Compressive strength tests are made on concrete trial mixes to evaluate the performance of available materials and to establish mix proportions that give the required strength. Strength tests are used also to control the quality of concrete being manufactured in the field. The compressive strength is defined as the average of the strengths of all cylinders of the same age made from a sample taken from a single batch of concrete. At least two cylinders, and preferably three, are required to constitute a test. Hence four or six specimens are required if tests are to be made at 7 and 28 days.

4-31. Preparation of Cylinders

a. The cylinders should be made as near as possible to the place where they will be stored for the first 24 hours. Sufficient concrete (about 1 cubic foot) for the desired number of cylinders must be available in the trial mix or field sample. Material must not be taken from the air content test, since this may be contaminated with excess water. The sample is taken from a stationary mixer by passing a receptacle completely through the discharge stream of the mixer at about the middle of the batch, or by diverting the stream into a container. The contents of a paving mixer should be discharged into a pile and sample material taken by a shovel from at least five different portions of the pile. Samples are taken from revolving drum truck mixers or agitators by repeatedly passing a receptacle through the entire discharged stream until sufficient concrete is collected into the pan (fig. 4-10). Thoroughly mix the sample before filling the cylinder molds.

b. Clean and dry the molds. Give the metal molds, if these are used, a light coat of oil before filling. Fill the molds in three equal layers, rodding each layer 25 times, as described for the slump test. Tap the side of the mold with the rod after each layer to close holes that may be left by the rod. Strike off the top surface level with the top of the mold. Store the cylinders on a level, flat, hard surface free from vibration. In moving the cylinders of fresh concrete in cardboard molds, lift them on a flat trowel to prevent bulging of the lower end of the mold. Tag the specimens or molds for identification.

c. Cover the specimens in the molds with polyethylene sheeting and place wet burlap over them (fig. 4-11). Maintain the temperature of the specimens between 60° F. and 80° F. (16° to 27° C.). Stoves or electric light bulbs may be used for heating. Cooling is accomplished, if needed, by evaporation of excess water from the wet burlap. A fan can be used to cause lower temperatures than will

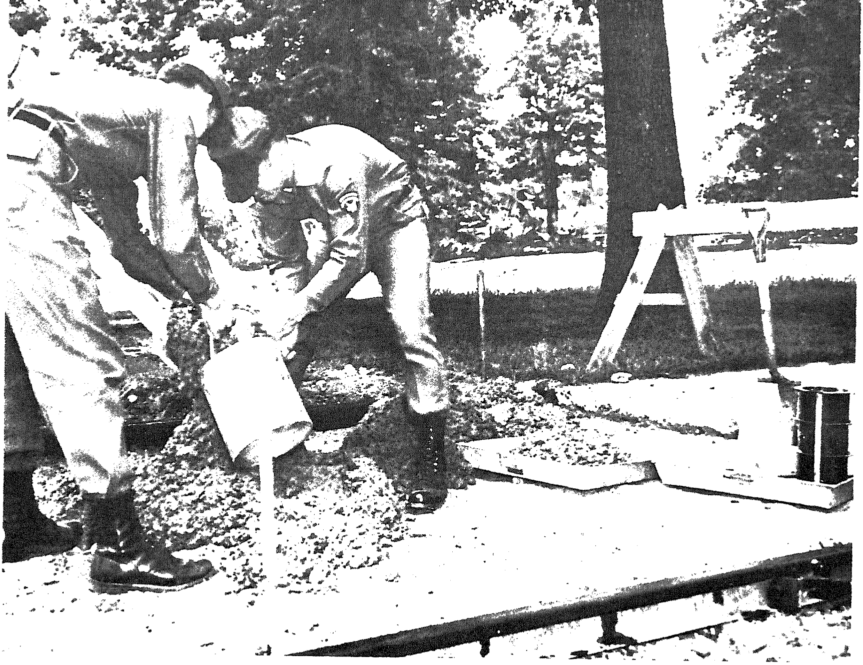


Figure 4–10. Sampling concrete from truck mixer.

occur from natural evaporation. Keep the burlap wet; if it is allowed to dry the concrete curing will be adversely affected. At the end of 24 hours, the specimens are removed from the molds and stored to continue curing until test. Mask the sides of the cylinders with wax or permanent ink, or attach moisture-proof tags with wire for identification. Individual handling of samples from different sources is as indicated:

(1) Specimens intended for checking the strength of laboratory trial mixes or to serve as the basis for acceptance or quality control of field concrete are moist cured at 70° to 76° F. (21° to 26° C.) until test. They are stored in moist rooms, damp sand, or sawdust or in lime water to maintain free water on all surfaces of the specimen at all times.

(2) Test specimens made in the field to determine when forms may be removed should receive, insofar as possible, the same protection from the elements on all surfaces as is given to the portions of the structure that they represent. They are stored in or on the structure as near as possible to the point of use. They are tested in the moisture condition resulting from the specified curing treatment. Speci-

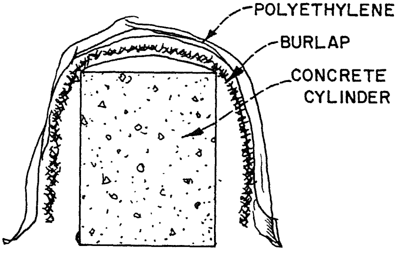


Figure 4–11. Curing of test cylinders

ture may be put into use are removed from the molds at the time of removing formwork from the structure.

(3) Specimens shipped from the field to the laboratory for testing are packed in a sturdy wooden box or other suitable container surrounded by wet sawdust or wet sand. Protection from freezing during storage or shipment must be provided. Upon receipt in the laboratory, moist curing is continued.

d. The ends of compression test specimens must be plane within 0.002 inch and within 3 degrees of being perpendicular to the axis of the cylinder.

(1) Specimens formed in strong metal molds having accurately flat base plates can be capped with neat cement at 2 to 4 hours after molding. A stiff paste or portland cement and water is made at the time the cylinder is molded, so that the capping mixture will have shrunk before application. Any free water or laitance (layer of fine particles on the surface) is removed from the end of the specimen. The paste is applied to the top of the concrete and worked with a flat plate until it is smooth and level with the top of the mold.

(2) Hardened concrete specimens may be ground to place ends or capped with a material having greater compressive strength than the concrete. Prepared mixtures of sulfur and granular materials, special high-strength gypsum plasters, and neat high-early strength cement are satisfactory capping materials (ordinary low-strength plaster of paris, or compressible rubber or fibrous materials are not suitable for caps). These materials are applied in a plastic state and finished to the desired plane surface by applying glass or metal plates and squeezing out excess material to provide a cap that is as thin as possible. Sulfur caps may be applied in time to harden for at least 2 hours before testing. Plaster caps cannot be stored over 4 hours in the moist room. Neat cement caps must be aged 3 days or more in the moist room. During capping, moist cured specimens should be protected against drying by covering them with wet burlap.

The procedure for conducting the compression test is as follows:

a. Prepare the testing machine (fig. 4–12) by cleaning the bearing plates and, if needed, cleaning and lubricating the spherical seat. Check the operation of the machine. Keep the specimens moist by covering with wet burlap during the period between removal from the curing environment and testing.

b. Determine the diameter of specimen to the nearest 0.01 inch by averaging two diameters

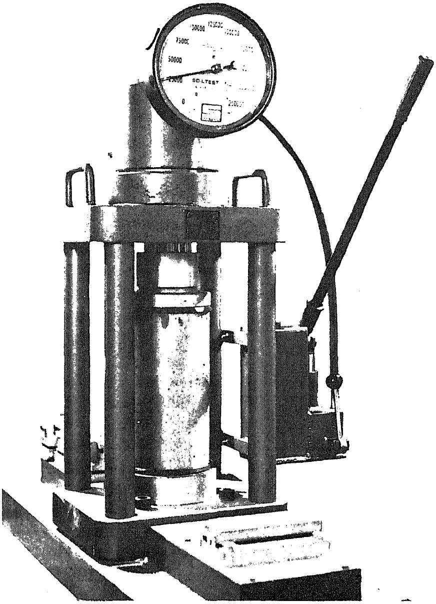


Figure 4–12. Compression testing machine with 6-inch diameter by 12-inch high cured concrete test cylinder in place.

easured at right angles to each other at mid-height
f the cylinder. Measure the length, including caps,
the nearest 0.1 inch. Record dimensions on a pre-
ared data sheet.

c. Place the specimen on the lower bearing block,
ring the upper block near to contact, and aline the
xis of the specimen with the center of thrust of the
pherical head. Carefully and slowly bring the
pherical head into contact with the specimen, rotat-
g the movable portion gently by hand so that uni-
orm seating is obtained. Apply the test load con-
inuously and without shock at a rate of 20 to 50
ounds per square inch per second. Observe and
ecord the maximum load during the test. Observe
ype of fracture and record any unusual features. The
ormal cone-type fracture is shown in figure 4–13.

d. The compressive strength of the concrete is the
aximum load divided by the cross sectional area of
he cylinder:

$$f'_c=\frac{4P}{\pi D^2}=1.234\,\frac{P}{D^2}$$

which f'_c is compressive strength, pounds per
quare inch

P is maximum load in test, pounds

D is average diameter of cylinder, inches

eport the identification, diameter and length, maxi-
um load, compressive strength, irregular fracture
r defect in caps or specimen, and age of specimen at
est. Report average compressive strength of all
ylinders from the same concrete sample.

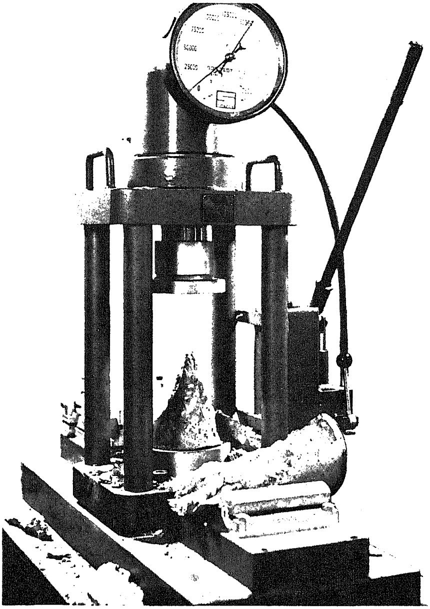


Figure 4–13. Normal fracture of concrete test cylinder in compression. Load of 154,000 pounds exerted over a 28.3 square-inch area determines the strength to be 5400 psi.

CHAPTER 5

SOIL STABILIZATION

Section I. MECHANICS OF STABILIZATION

5–1. Discussion

a. Soil stabilization is the process which alters any property of the soil to improve its engineering characteristics. Stabilization may be divided into two categories: cementing action and modification. Cementing action results in a hardened product of considerable strength. Modification results in an improvement of some soil properties or in the elimination of some undesirable features in a soil. The end product may or may not show a significant increase in strength. Stabilization cannot always be classified into one or the other category exclusively. It is possible to achieve a thoroughly cemented end product by adding a sufficient quantity of stabilizing agent. However, basic improvement can be achieved many times with a much smaller quantity of cementing agent and final stabilization completed with modification.

b. Soil stabilization is used to provide improved subgrade and base course characteristics for pavements. Stabilized soils can often be adequate for light traffic pavements, and for parking and storage areas where an all-weather surface is required yet traffic does not justify a higher strength pavement. Surface treatments are used also to provide control of dust.

5–2. Stabilization by Cementing Action

This category requires the addition of agents to the soil to produce the hardened product. There are four main stabilizing agents which can be added, and the method of treatment bears the name of these agents; soil-cement, soil-lime, lime-fly ash, and bituminous (soil-asphalt). The first three methods have much in common and involve somewhat similar construction practices. They depend upon hydration, pozzolanic action of lime with silica and alumina, alteration of the clay material, or a combination of these actions. The result is a semirigid fairly brittle material with considerable compressive strength and moderate flexural strength when tested either statically or dynamically. The ultimate strength depends to a great degree on the density which is achieved during compaction and before the mix cures. In bituminou treatment, the end product performs differently. A least initially, the product is much less brittle and it behavior depends on the nature of the loading (stati or dynamic), and the temperature when the load i applied.

5–3. Stabilization by Modification

Soil stabilization by modification usually results i something less than a thoroughly cemented hardene or semihardened material. This type of stabilizatio may be accomplished by compaction, by mechanica blending, by adding cementing materials in smal amounts, or by addition of chemical modifiers. Cement and lime modifiers (cement modified soil an lime modified soil) are used in quantities too smal to provide high strength cementing action. They reduce the plasticity of clay soils. Calcium chloride o sodium chloride are added to the soil to retain moisture (and hence control dust), to hold fine materia for better compaction, and to reduce frost heave b lowering the freezing point of water in the soil. Bituminous materials, such as cutback asphalts or asphaltic penetrative soil binder (APSB), and certai chemicals, such as polyvinyl acetate emulsion (DCA 70), are used to waterproof the soil surface and t control dust.

5–4. Stabilization Processes

The method of soil stabilization chosen is determine by the amount of stabilizing required and the conditions encountered on the project. An accurate soil description and classification (paras 2–3 and 2–8) i essential to the selection of the correct materials an procedure. In table 5–1 are listed the most suitabl treatments for various soil types to stabilize these soi for different objectives. Table 5–2 is a summary o soil stabilization methods. More extensive discussio of field construction operations to stabilize soils ma be found in TM 5–330 and DA Pam 525–5.

Purpose	Soil type	Method
1. Subgrade stabilization a. Improved load carrying and stress distribution characteristics.	Fine granular	SA, SC, MB, C
	Coarse granular	SA, SC, MB, C
	Clays of low PI	C, SC, CMS, LMS, SL
	Clays of high PI	SL, LMS
b. Reduce frost susceptibility	Fine granular	CMS, SA, SC, LF
	Clays of low PI	CMS, SC, SL, LMS
c. Waterproofing and improved runoff	Clays of low PI	CMS, SA, LMS, SL
d. Control of shrinkage and swell	Clays of low PI	CMS, SC, C, LMS, SL
	Clays of high PI	SL
e. Reduce resiliency	Clays of high PI	SL, LMS
	Elastic silts or clays	SC, CMS
2. Base course stabilization a. Improvements of substandard materials	Fine granular	SC, SA, LF, MB
	Clays of low PI	SC, SL
b. Improved load carrying and stress distribution characteristics.	Course granular	SA, SC, MB, LF
	Fine granular	SC, SA, LF MB
c. Reduction of pumping	Fine granular	SC, SA, LF, MB, membranes
3. Dust palliative	Fine granular	CMS, SA, Oil or bituminous surface spray, APSB
	Plastic soils	CMS, SL, LMS, APSB, DCA 70

Where the methods of treatment are:

APSB=Asphalt Penetration Surface Binder
C=Compaction
CMS=Cement modified Soil
DCA70=Polyvinylacetate emulsion
LF=Lime Fly ash
LMS=Lime Modified Soil
MB=Mechanical Blending
SA=Soil-Asphalt
SC=Soil-Cement
SL=Soil-Lime

Table 5-2. Summary of Stabilization Methods

a. Cementing agents (increased strength through action)

Cementing agent		Portland cement				Lime (hydrated and quick)			Bituminous (with asphalt or tar)			
Primary Use		Base, subbase, shoulders				Subbase and shoulder occational base			Base, subbase, shoulders			
	Quantity of additive By weight	3-8	5-9%	9-12%	10-16		2-6	2-8	3-6	3-6	5-9	
Relative effect of soil types	Coarse granular	A				N A[1]			A			
	Fine granular		A			N A[1]				A		
	Fine low PI			A			A				B	
	Grained Hi PI				B			A				N A
Relative change in soil properties	Compressive strength	LI	LI	LI	LI	----	LI	LI	MI[2]	MI[2]	SI[2]	----
	Flexural strength	LI	LI	LI	MI	----	MI?	MI?	MI[2]	MI[2]	SI[2]	----
	Durability	LI	LI	LI	MI	----	LI	LI	LI	LI	MI	----
	Density	SI	SI-NC	NC-SD	SD	----	SD	MD	SI-SD	SI-SC	SD	----
	Water absorption	SD	SD	LD	MD	----	LD	LD	LD	LD	LD	----
	Water retention	SD	SD	LD	MD	----	LD	LD	LD	LD	LD	----
	Permeability	LD	LD	SD	NC-SI	----	SD-SI	SI	LD	LD	SD	----
	Volume change	LD	LD	LD	MD	----	LD	LD	MD	MD	MD	----
	Plasticity	MD	MD	MD	LD?	----	LD	LD	SI?	SI	SD?	----

Table 5-2. *Summary of Stabilization Methods — Continued*
a. Cementing Agents (increased strength through action) — Continued

Lime fly ash			
Base, subbase, shoulders			
3-5% 10-20%	Lime fly ash	5-9% 10-25%	Lime fly ash
A			
	A		
		B	
			N/A
LI	LI	MI	----
LI	LI	MI	----
LI	LI	MI	----
SD	SD	SD	----
SD	SD	LD	----
SD	SD	LD	----
LD	LD	SD	----
LD	LD	LD	----
MD	MD	LD	----

Table 5-2. Summary of Stabilization Methods — Continued

a. Cementing Agents (increased strength through action) — Continued

	Cement	Lime (hydrated & quick)	Bituminous (with asphalt or tar)	Lime flyash
Primary use	Base, subbase, shoulders	Subbase & shoulders	Base, subbase, shoulders	Subbase & shoulders
Construction procedure	Pulverize, mix, compact, cure. For best results, use plant mix and granular aggregate.	Pulverize, mix, compact and cure.	Batch Plant or hot pen. Mix, cure, compact.	Pulverize, mix, compact cure. A sup. base req. Plant Mix operation and granular aggregate.
Limitations	Organic soils, some fat clays, sand and gravels of highly uniform grain size and acidic podzolic sands do not respond to treatment.	Gravel and sand soils with less than 25% finer than the #200 sieve do not respond to treatment. Quicklime in caustic.	Creeps under high temperature or sustained load; cutback or emulsions require warm temp for curing.	Develops strength slowly; requires warm temperatures (50° F.+) for long duration to cure sufficiently.
Unique advantages	Develops high strength more rapidly than lime or lime fly ash products.	Construction expedient; quickly lowers PI of clay. Moderate delay between mixing and compaction not critical.	Waterproofing effect	Cure can be interrupted and restarted without adverse effects. Speed not essential

Table 5-2. Summary of Stabilization Methods — Continued

b. Modifiers

Primary function			Increase stability						
Stabilization method			Compaction				Mech blending of soils		
Primary use			Base, subbase, subgrade, shoulders				Base and subbase		
Quantity of additives required by weight									
Rel. effect on soil types	Coarse granular		B				A		
	Fine granular			A-WG B-PG				A	
	Fine Grained Soils	Low PI			A				N/A
		High PI				B			N/A
Relative change	Compressive strength		MI	MI	MI[3]	MI[3]	MI	MI	---
	Flexural strength		SI	SI	SI[3]	SI[3]	SI	SI	---
	Durability		NC	NC	MI[3]	MI[3]	NC	NC	---
	Density		I	I	I	I or D	LI	LI	---
	Water absorption		NC	NC	SD	SD	NC[4]	NC[4]	---
	Water retention		NC	NC	SD	SD	NC-SI	NC-SI	---
	Permeability		SD	SD	SI-LD	SI-LD	SD-MD	SD-MD	---
	Volume change		NC	NC	SD[3]	MD[3]	NC	NC	---
	Plasticity		NC	NC	NC	NC	NC[4]	NC[4]	---

Table 5-2. Summary of Stabilization Methods — Continued
b. Modifiers — Continued

Reduce plasticity volume change and resilience							Waterproof		
	Cement			Lime			Bitumens		
Subgrade, subbase,			Subgrade, subbase			Subbase, subgrade			
	1-4%	2-6%		1-4%	2-6%	1-4%	1-4%	1-4%	
N/A			N/A			B			
N/A			N/A				A		
	A			B				B	
		B			A				N/A
---	MI	SI	---	MI	MI	MC	SI	SI	---
---	MI	SI	---	MI	MI	MC	SI	SI	---
---	MI	SI	---	MI	MI	SI	MI	MI	---
---	NC-SD	NC-SD	---	SD	SD	SI	SI	SI-SD	---
---	MD	MD	---	MD	LD	SD-SI	MD	MD	---
---	MD	MD	---	MD	MD	SI-SD			---
---	SD-SI	SI	---	SD-SI	SI	LD	LD	SD	---
---	MD	MD	---	MD	LD	NC	NC	SD	---
---	MD	MD	---	MD	LD	SI?	SI?	SD?	---

Table 5-2. Summary of Stabilization Methods – Continued
b. Modifiers – Continued

Construction procedure	Vibration equipment and steel rollers,	Sheepsfoot and pneumatic comp w. aeration and water addition varied as req.	Mix thoroughly with field equipment or batch plants, then compact.	Pulverize, mix, and compact.	Pulverize, mix, compact, and cure.	Mix, cure and compact.
Limitations	Ultimate change in properties small compared to that achieved by some stabilizers.		Sufficient modification of detrimental soil may be costly	Cost may not be justified by modest property improvement.	Cost may not by justified by modest property improvement	Cost may not be justified by modest property improvement
Unique advantages.	Low cost and ease of control		Can make unsuitable soil suitable.	Construction expedient; improve unsuitable soils.	Construction expedient; improve unsuitable soils.	Helps maintain "As Constructed" condition.

Legend:

A	-Most suitable soil type for given stabilization method - significant property improvements	M	-Moderate
B	-Moderate property improvements	S	-Small
N/A	-Not suitable for use with given soil	?	-Uncertain
WG	-Well graded soils	I	-Increase
PG	-Poorly graded	D	-Decrease
L	-Large	NC	-No change

Notes:

[1] Not suitable unless granular material contains excess plastic fines.
[2] Cementing effect and resulting strength increase greatest for low temperature and dynamic loads.
[3] Provided water content, density, and compaction method are properly controlled.
[4] Unless increase desired through additives of fines.

Section II. SOIL-CEMENT STABILIZATION TESTS

5–5. Soil-Cement Tests

Soil-cement stabilization consists of mixing soil, cement, and water in definite proportions and compacting the mixture to a prescribed density. The tests are performed to determine three fundamental requirements: the amount of cement to be added, the amount of water to be added, and the density to which the soil-cement must be compacted.

a. Description. There are three standard tests: moisture-density, wet-dry, and freeze-thaw; a shortcut test for sandy soils; and a rapid test procedure used for soil-cement stabilization. The moisture-density test determines the proper (optimum) moisture content and maximum density for molding laboratory specimens. In the field, this test is used to determine the quantity of water to be added and the density to which the mixture should be compacted. The wet-dry test indicates whether the hardened soil-cement will stay hard or soften from exposure to moisture variations. The freeze-thaw test not only shows how soil-cement reacts to weather, but also whether the cement has hardened the soil or not. Shortcut test procedures for sandy soils do not involve a series of new tests, but rather use previous test information to reduce the testing required. The rapid test procedure is adequate for emergency construction and for small projects where more complete testing is impractical.

b. Selection of Cement Contents. Specimens prepared with very high cement contents will all pass the tests and a minimum cement factor cannot be determined. On the other hand, inadequate cement contents will cause all specimens to fail the test.

(1) The principal requirement of a hardened soil-cement mixture is to withstand exposure to the elements. Although strength is a requirement also, most soil-cement mixtures which possess adequate resistance to the elements also possess adequate strength. The reverse is not always true. *For example,* a compacted, clay loam soil (without added cement) can withstand relatively heavy loads. When mixed with cement and compacted, the same soil will have a stability equal to that of the raw soil (before the cement hardens). Under the effects of moisture, the original soil (without cement) will lose stability even though compacted. The soil with cement, upon hardening, will show an increase in stability.

(2) Economy is another important factor. Experience shows that approximately 85 percent of the soils considered for soil-cement stabilization can be adequately stabilized by adding 14 percent or less of cement. However, more than 50 percent require only 10 percent or less of cement. Selecting the optimum amount can result in a sizable saving of cement.

(3) As a general rule, the cement requirement increases as the silt and clay content increases, and a gravelly or sandy soil will require less cement. There is one exception. A poorly graded, one-size-particle sand with no silt and clay will require more cement than a sandy soil with some silt and clay. Well graded mixtures, containing gravel, coarse and fine sands, and with or without small amounts of silt or clay, require 5 percent or less of cement by weight. The poorly graded, one-particle-size sand requires about 9 percent by weight, while the other sandy soils only 7 percent. Nonplastic or moderately plastic silty soils require 10 percent and plastic clays 13 percent or more. The following listing indicates the usual range in cement requirements. Where classification appears in more than one category, plasticity and organic content are considerations. The cement contents are used as preliminary estimates which are verified or modified as the test data become available.

*Soil classification**	*Cement required (percent by weight)*
GW, GP, SW, SP, GM, or SM	3–5
SP, GM, SM, or GP	5–8
SM, SC, some GM or GC	5–9
SP	7–11
CL or ML	7–12
ML, MH, or OH	8–13
CL, CH	9–15
OH, MH, or some CH	10–16

*See chapter 2 for description of soil classifications.

c. Preparation for Testing. Two methods of testing will be described. The first is used with soils containing material that is retained on the No. 4 sieve (up to a maximum of 45 percent), and the second for soils not containing material that is retained on the No. 4 sieve. The maximum size particle used in the test specimen is ¾ inch. If larger particles are present in the sample, they are replaced with an equivalent weight of material from the portion that passes the ¾-inch sieve and is retained on the No. 4 sieve (para 2–64*b*). About 75 to 100 pounds of soil are prepared for testing. When necessary, the sample is dried until it is friable under a trowel. This may be accomplished by air-drying or oven-drying at a temperature of 60° C. (140° F.). The soil is then separated on the 3-inch, ¾-inch, and No. 4 sieves. All clusters should be broken up into individual particles. The soil passing the No. 4 sieve should be well mixed and stored in a covered container throughout the duration of the test. The material larger than the 3-inch sieve is not included in the calculations of grain-size distribution, but its quantity is noted and the material discarded. The material retained on the ¾-inch and No. 4 sieves is weighed and the weights recorded. They will be used in the computations. The material retained on the ¾-inch sieve is replaced with the material retained on the No. 4 sieve.

Before starting this test, it is necessary to select the cement contents that will be used in the freeze-thaw and wet-dry tests. The cement contents are usually selected in 2-percent increments to encompass the values given in paragraph 5–5.

For example, the table lists 7 to 11 percent for the SP sand. The cement content increments would be 7, 9, and 11 percent. In the CL–CH group (9–15 percent), the selected increments would be 10, 12, and 14 percent. Since maximum density varies only slightly with variations in the cement content, only the median value is used in preparing specimens for the moisture-density test. The procedures for determining the optimum moisture content (OMC) are similar to those described in paragraphs 2–63 to 2–66 with the following exceptions. Compaction is performed on five layers of approximately equal thickness to result in a total compacted depth of 5 inches. Each layer is compacted by 25 uniformly spaced blows of a 10 pound tamper (2 inch diameter face) dropped from a height of 18 inches. The computations and OMC determination then follow the procedures described in paragraph 2–65.

5–7. Wet-Dry and Freeze-Thaw Tests

After determining the maximum density and OMC, specimens must be molded for the wet-dry and freeze-thaw tests. These specimens are prepared using the computed OMC and the cement contents described above for the different soil classifications. The cement contents are selected in 2 percent increments either side of the median value. Two specimens are molded for each of the three cement contents; one for the wet-dry test, and one for the freeze-thaw test. The same procedure is used to mold the specimens as was used for the OMC determination. Special care must be used to scarify the surfaces between layers to assure a good bond. When the second layer is being placed, a 750 gram sample should be taken for a moisture determination. The molded specimens are placed in a moisture cabinet in an atmosphere of high humidity for 7 days to permit cement hydration before testing.

a. Conducting the Wet-Dry Test. After this 7 day curing period, the specimens are submerged in tapwater at room temperature for a period of 5 hours and removed. Then the specimens are dried in an oven at 71° C. (160° F.) for 42 hours and removed. The entire surface area is wire brushed to remove all material loosened during wetting and drying. Use two firm strokes on each portion of the surface. These strokes are applied full height and width with a 3 pound force. Five hours of water immersion, 42 hours of drying, and 1 hour of handling constitutes one cycle. The operation is repeated for a total of 12 cycles. After 12 cycles of test, the specimens are dried to determine the oven-dry weights.

b. Conducting the Freeze-Thaw Test. After the curing period, water-saturated felt pads about 1/4 inch thick, blotters, or similar absorptive materials are placed between the specimens and specimen carriers, and the assembly is placed in a freezing cabinet with a constant temperature not warmer than −23° C. (−10° F.) for 24 hours and removed. The assembly is then allowed to thaw in a moist room or in suitable covered containers with a temperature of 21° C. (70° F.) and a relative humidity of 100 percent for 23 hours. Free water is made available to the absorbent pads to permit the specimens to absorb water by capillarity during the thawing period. The specimens are next given two firm strokes on all areas with the wire brush to remove material loosened during freezing and thawing. Scale that has formed should be removed, with a sharp pointed instrument if necessary. Twenty-four hours of freezing, 23 hours of thawing, and 1 hour of handling (total 48 hours) constitutes one cycle. After being brushed at the end of each thawing period, the specimens are turned over end-for-end before they are replaced on the water-saturated pads. The test is continued for a total of 12 cycles after which the specimens are dried to constant weight at 110° C. (230° F.) and weighed to determine their oven-dry weights.

5–8. Comprehensive Strength Test

As mentioned previously, compressive strength is a supplementary consideration to the resistance to weather. Five specimens are molded, for each of the three cement contents (para. 5–7), using the optimum moisture and the molding procedure of the moisture-density test. If a 4 inch diameter, 4.6 inch high specimen is to be molded (a 2.0 inch diameter, 4.0 inch high specimen is also acceptable), only the samples with material retained on the No. 4 sieve (up to 45 percent maximum) are used. Compressive strength tests are conducted after 2, 7, 14, 21, and 28 days (tests at 14 and 21 days are optional). Four hours prior to the test, one specimen at each cement content is withdrawn from the moisture cabinet and immersed in tapwater at room temperature (approx. 70° F.). Each specimen is tested in the compression machine until failure and the total load required for failure is recorded. The compressive strength in pounds per square inch is computed by dividing the total load by the average area of the top and bottom of the specimen.

5–9. Calculations and Criteria

The results of the wet-dry and freeze-thaw cycles are indicated as soil-cement losses. These losses are computed by using the original dry weights and final corrected dry weights.

a. Water-Of-Hydration Correction. The final oven-dry weight of the specimen includes some water used for cement hydration that cannot be driven off at 110° C. (230° F.). The average amount of this water retained in the specimen is based on the type of soil; gravels 1½ percent ±, sands 2½ percent ±, silts 3 percent ±, and clays 3½ percent ±. This correction is computed by the following formula:

Corrected oven-dry weight

$$+\frac{\text{measured oven-dry weight of specimen}}{\text{percent water retained}+100}\times 100$$

Example: Sample composed mostly of sand weighs 3.77 pounds at the end of the test. Water of hydration is 2.5 percent.

Corrected oven-dry weight=

$$\frac{3.77}{2.5+100}\times 100=\frac{3.77}{1.025}=3.68 \text{ lb}$$

b. Soil-Cement Loss. The soil-cement loss can now be calculated, as a percentage of the original dry weight, or:

Soil-cement loss=

$$\frac{\text{original oven-dry weight}-\text{final corrected oven-dry weight}}{\text{original oven-dry weight}}\times 100$$

Example: The sample in the example above weighed 3.99 pounds originally.

$$\text{Soil-cement loss}=\frac{3.99-3.68}{3.99}\times 100=7.8\%$$

This value would be reported to the nearest whole number or as 8 percent.

c. Weight Loss Criteria. The minimum cement content recommended for use is the one for which losses of specimen weight during 12 cycles of the wet-dry test or freeze-thaw conform to the following standards:

GW, GM, GC, SW, SM, SC, and SP soils, not over 14 percent.

ML, and MH soils, not over 10 percent.

CL, CH, OH, and Ol soils, not over 7 percent.

d. Strength Criteria. The strength of soil cement specimens tested in compression at various ages should increase with age and with increases in cement, in the ranges of cement contents producing results meeting the requirements above. A sample that has an unconfined compression strength of approximately 300 p.s.i. after curing 7 days and shows increasing strength with age can be considered adequately stabilized.

e. Cement Weight-to-Volume Conversion. The required cement content by weight must be converted to the equivalent cement content by volume for control during construction, since this is the easier quantity to use in the field. The following formula illustrates the calculation.

$$\text{Volume of cement (percent)} =\frac{D-(D/C)}{94}\times 100$$

Where:

D=oven-dry density of soil-cement (lb/cu ft)

$$C=\frac{100+\text{percent cement (by weight)}}{100}$$

94=weight of 1 cu ft of cement.

The nomograph in figure 5–1 makes the conversion without computation. Use a straightedge placed at the soil-cement density and at the percent by weight of cement. Read the percent cement by volume on the right-hand scale.

5–10. Modified Mix Design for Sandy Soils

Sandy soils are generally the most readily and economically stabilized becaues they require the least amount of cement for adequate hardening and because they contain a minimum amount of material which prevents intimate mixing of soil and cement. The following shortcut testing procedures for sandy soils will not always indicate the minimum cement contents required but the results will be close enough to be on the safe side and economical. If time permits, the procedures in paragraph 5–9 are followed for greater economy of design.

a. Two procedures are used; one for soils not containing material retained on the No. 4 sieve, and the other for soils containing material retained on the No. 4 sieve. The procedures can be used only with soils containing less than 50 percent of material smaller than 0.05 mm (silt and clay) and less than 20 percent smaller than 0.005 mm (clay). Dark gray to black sandy soils obviously containing appreciable organic impurities, together with miscellaneous granular materials such as cinders, caliche, chat, chert, marl, red dog, scoria, shale, and slag should be tested using the full procedures and not tested by the modified methods for sandy soils. When coarse grained or sandy soils (generally of groups GW, GP, GM, SW, or SM) are encountered, they may be classified for testing purposes using either the first or the second procedure. There is one other exception. Granular soils with materials retained on the No. 4 sieve whose bulk specific gravity is less than 2.45 cannot be tested.

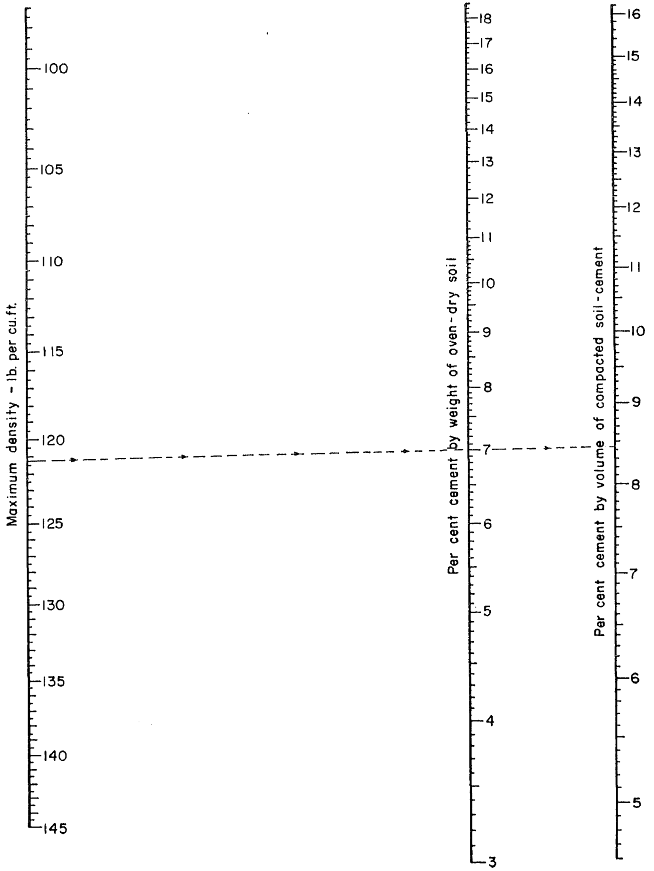


Figure 5–1. Relation of cement content by weight of oven-dry soil to cement content by volume of compacted soil-cement mixture.

b. The procedure consists of the following sequence:

(1) Determine the soil gradation.

(2) Determine the bulk specific gravity of the material retained on the No. 4 sieve. Procedure is described in paragraph 2–35.

(3) Perform the moisture-density test of an estimated soil-cement mixture.

(4) Locate the indicated cement requirements from the charts.

(5) Perform compressive-strength tests to verify the cement requirement.

c. The step-by-step method for soils with no material retained on the No. 4 sieve is as follows:

(1) Determine the maximum density and OMC

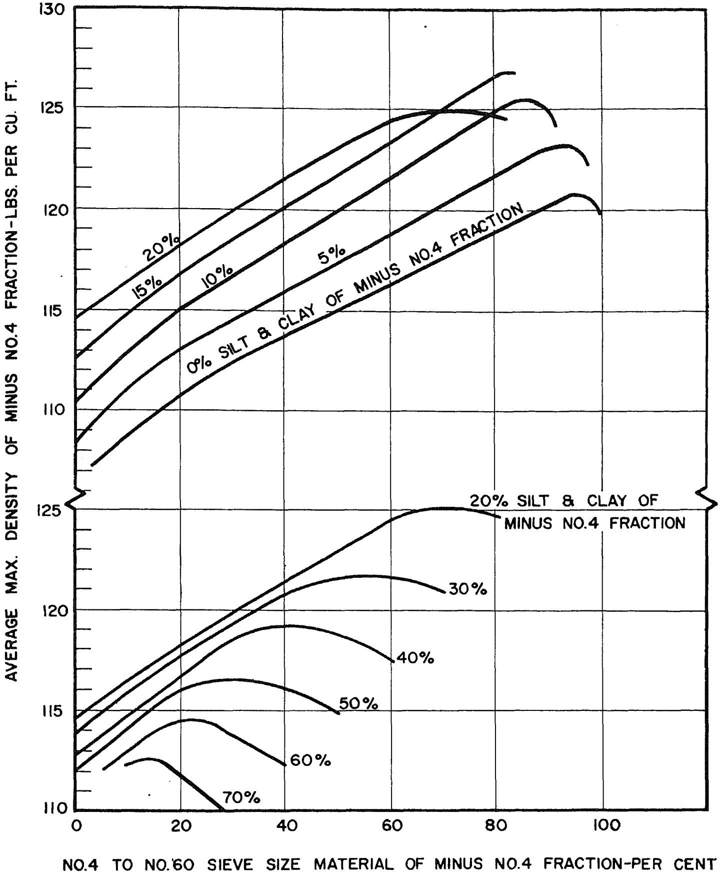


Figure 5–2. Average maximum densities of the minus No. 4 fraction of soil-cement mixtures.

for a mixture of soil and cement. (Fig. 5–2 will give an estimated maximum density. This value and the percentage of material smaller than 0.05-mm are used with fig. 5–3 to determine an estimated ce content.)

(2) Use the maximum density value

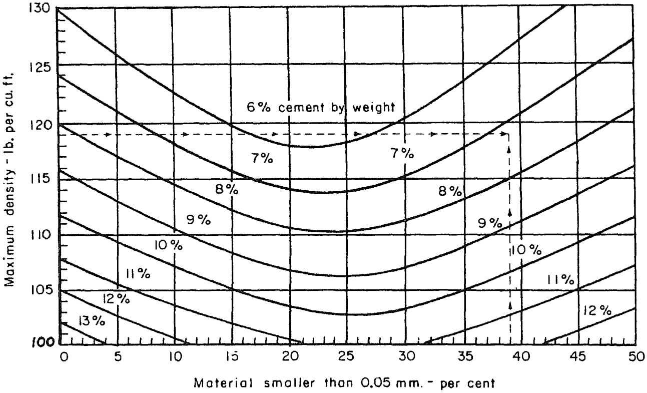


Figure 5–3. Indicated cement contents of soil-cement mixtures not containing material retained on the No. 4 sieve.

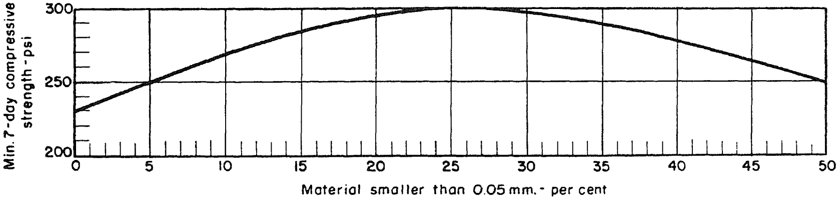


Figure 5–4. Minimum 7-day compressive strengths required for soil-cement not containing material retained on the No. 4 sieve.

above) and figure 5–3 to determine an indicated cement requirement.

(3) Mold three compressive-strength specimens at maximum density and OMC.

(4) Moist-cure the specimens for 7 days and test for strength.

(5) Plot the value of the averaged compressive strength on figure 5–4. If this plot is above the curve, the cement factor is probably too low and needs adjusting. Two new test specimens are prepared; one at the cement content as computed in (2) above, and the second with a 2 percent higher cement content. The full freeze-thaw test is performed on these two specimens.

d. The method of testing of soils containing material retained on the No. 4 sieve is as follows:

(1) Determine the maximum density and optimum moisture content for a mixture of soil and cement. Use figure 5–5 for an estimated maximum density, and figure 5–6 with this density, percentage of material retained on the No. 4 sieve, and percentage smaller than 0.05-mm to determine cement content. The 45 percent maximum retained on No. 4 sieve (discussed in para 5–8) still applies. Also, any material larger than ¾ inch must be replaced with an equivalent weight of the material passing the ¾-inch sieve and retained on the No. 4 sieve.

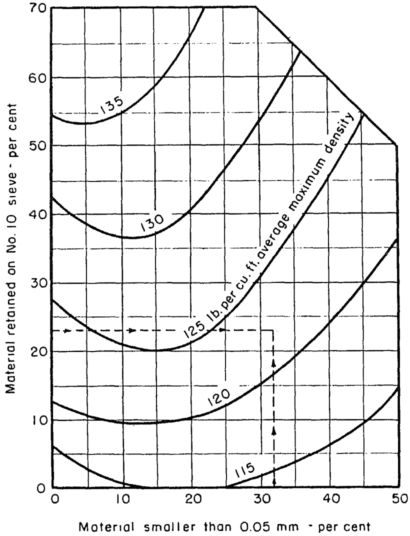


Figure 5-5. Average maximum densities of soil-cement mixtures containing material retained on No. 4 sieve.

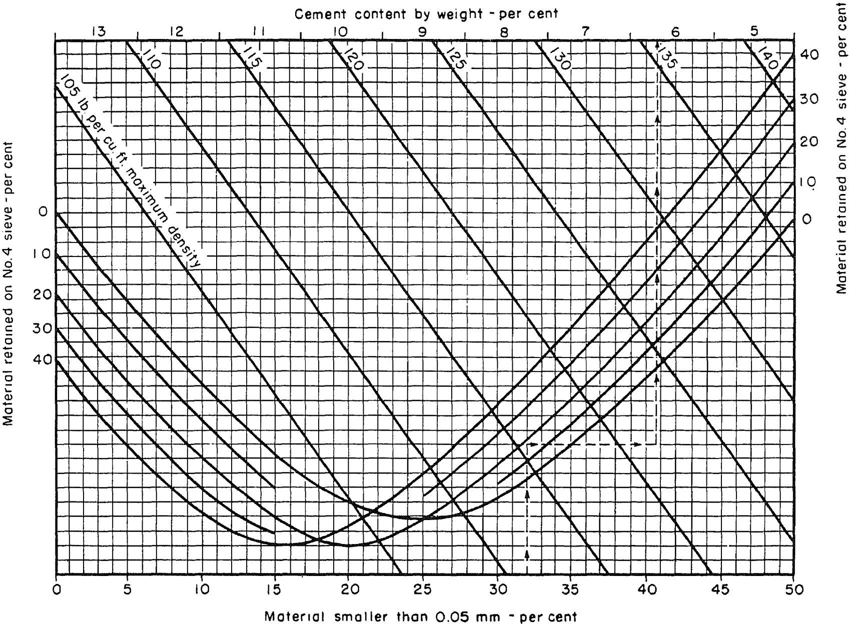


Figure 5-6. Indicated cement contents of soil-cement mixtures containing material retained on No. 4 sieve.

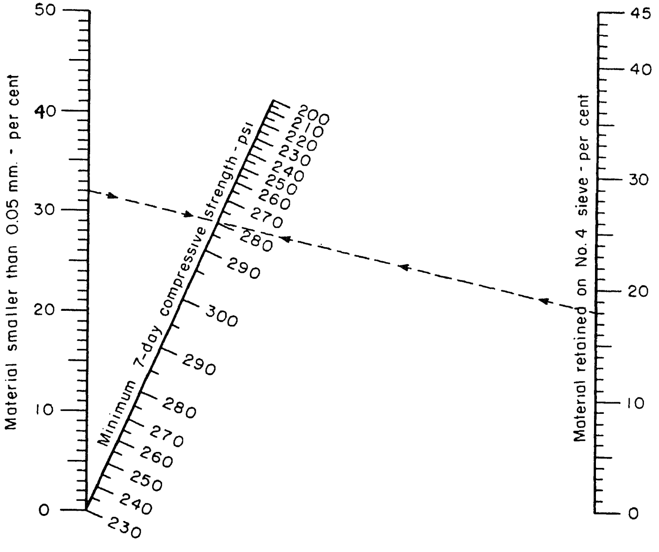


Figure 5-7. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on No. 4 sieve.

(2) Using the maximum density from (1) above and figure 5–6, determine the indicated cement requirement.

(3) Mold test specimens at maximum density and OMC.

(4) After 7 days moist-curing (para 4–6) test for compressive strength and average.

(5) Using figure 5–7, determine the allowable compressive strength for the soil-cement mixture. Connect points on the right- and left-hand scales of the nomograph and read the minimum required compressive strength from the inclined center scale. If the tested strength is equal to or greater than the allowable, the cement content is adequate. If the strength is too low, the cement factor is also too low, and a full test should be performed.

5–11. Approximate and Rapid Tests

a. Squeeze Test. A sample mixture of soil, cement, and water may be tested for optimum moisture content by squeezing some in the hands. When squeezed firmly, the sample at optimum moisture content will form a cast that will cling together and moisten the hands but water cannot be squeezed out of the mixture.

b. "Pick" and "Click" Tests. Specimens covering a wide range of cement contents (*for example:* 10 percent, 14 percent and 18 percent) are molded at optimum moisture and maximum density. After at least 1 or 2 days of hardening while kept moist, and after a 3-hour soaking period, the specimens are inspected by "picking"—done with a pointed instrument such as a dull ice pick or bayonet—and by sharply "clicking" each specimen against a hard object such as concrete or another sound specimen, to determine their relative hardness when set. If the specimen cannot be penetrated more than ⅛ to ¼ inch by "picking" and if it produces a clear or solid tone upon "clicking" an adequate cement factor is indicated. When a dull thud or "punky" sound is obtained, there is inadequate cement even though the specimen may resist picking. The age of the specimens is definitely a factor, and a specimen which may not test properly at first try may harden properly a few days later. Some satisfactory specimens require 7 days or longer to produce adequate hardening. The test results will indicate the proper content. If the results show that some intermediate content may be satisfactory, new test specimens (at the suggested content) should be prepared and tested. It is important to remember that too much cement is not harmful (although more expensive), but that too little will not produce a satisfactory stabilization.

Section III. BITUMINOUS STABILIZATION TESTS

5–12. Description

Bituminous soil stabilization refers to a process by which a controlled amount of bituminous material is thoroughly mixed with an existing soil or aggregate material to form a stable base or wearing surface. Bitumen increases the cohesion and load-bearing capacity of the soil and renders it resistant to the action of water. Bitumens used include asphalt cutbacks, asphalt emulsions, and tar. The type of soils most readily stabilized with bituminous materials are granular friable, contain small amounts of clay and silt, and are free of mica, organic material, and other injurious substances. Gradation and plasticity requirements for soils to be stabilized with bituminous materials are as follows: More than 50 percent passing the No. 4 sieve and less than 35 percent passing the No. 200 sieve; liquid limit less than 25, and PI less than 5. Asphalt-treated base courses should have no more than 12 percent passing the No. 200 sieve with a PI of the passing No. 40 fraction of less than 10.

a. Type of Bituminous Material. Generally the heaviest grade of bitumen which can be readily worked into the soil is employed. This will depend upon the soil, the available mixing equipment, and the weather at the time of construction. The RC grades work best and are recommended for use in the extremely sandy soils or those containing a minimum of silt and clay particles. As the silt and clay particles become more prevalent in the sandy soil, a medium curing cutback is advised. In soils that have as much as 30 to 40 percent silt and clay particles, slow-curing asphalts have been successfully used, particularly in arid regions. The following types of bituminous material have been successfully used:

(1) Asphalt cutback.

(*a*) Rapid-curing grades, RC–70 to RC–800.

(*b*) Medium-curing grades, MC–70 to MC–800.

(*c*) Slow-curing grades, SC–70 to SC–800.

(2) Slow-setting emulsions.

(3) Tars of grades RT–3 to RT–7.

b. Quantity of Bitumen. Bitumen contents in the range of 3 to 9 percent, exclusive of volatile materials, are generally used for soil stabilization with increasing fines in the soil requiring the higher bitumen contents. The quantity to be employed is first estimated from the results of the soil analysis. If no tests are to be used to determine the bitumen content, ½ gallon of liquid bitumen per square yard per inch of compacted depth should be specified as a starting point for trial mixing, corrections to be made to trial mixes as deemed necessary. If testing is to be used for determining actual bitumen percentages, the test procedures of paragraph 5-7 should be followed, testing the estimated content and a range of bitumen contents above and below the estimated value at ½- to 1-percent intervals. These estimated percentages are actual bitumen contents, exclusive of volatile materials, based on the material finer than the No. 4 sieve in the soil. Additional bitumen, approximately 3 percent by dry weight of the plus No. 4 portion of the soil, is required for wetting the coarser material. The total bitumen content to be specified is determined from the following formula:

Percent bitumen specified = (percent bitumen determined by test) × (percent of minus No. 4 material) + (3 percent of plus No. 4 materials).

5-13. Method of Testing Mixtures

This method (ASTM Designation D 915) provides for determination of water absorption, expansion, and extrusion characteristics of compacted soil or soil-aggregate mixtures containing liquid bituminous materials. The method may be used for determining the characteristics of a mixture of specified proportions under specified conditions of curing or noncuring. Also, it may be used for determining the effects on these characteristics of varying the curing and the proportions of the different ingredients. The test results are not intended for use in formulas to determine thickness nor to predict relative field performance of the different bituminous materials.

a. Apparatus. The apparatus will consist of the following:

(1) *Mechanical mixer.* Capable of producing intimate mixtures of soil, water, and bituminous material.

(2) *Compaction apparatus.* Specified in the *Tentative Method of Testing Soil-Bituminous Mixtures*, ASTM Designation D 915 consists of the following:

(*a*) *Compaction mold.* A cylindrical hardened steel mold having an inside diameter of 2.000 ± 0.001 inch, a length of 5.25 inches, and an outside diameter of 2.5 inches or greater. The mold is equipped with a suitable device to control the specified drop and to protect the top of the mold during precompaction with the tamper.

(*b*) *Compaction plungers.* One hardened steel plunger having a circular face, 1.997 ± 0.001 inch in diameter and a length of 5.5 inches; and one hardened steel plunger having a diameter of 1.997 ± 0.001 inch and a length of 2.5 inches.

(*c*) *Tamper* (*blade type*). The tamper is made of steel and has an overall length of 14½ inches. For the upper 6 inches, it is turned to a steel shank ⅜ inch in diameter and fitted with a wooden handle 1⅜ inches outside diameter. The flat blade is 8½ inches in length, 1⅜ inches in width next to the handle, and 1⅞ inches in width of the tamping end. The thickness of the flat blade is ⅝ inch at the handle and for a distance of 6½ inches below the handle. The lower 2 inches of the blade is ¼ inch in thickness with the tamping edge finished to a rounded edge. The total weight of the tamper is approximately 1200 grams.

NOTE

> The inside diameters of all compaction and extrusion test apparatus and the diameter of the testing end of the test plunger should be checked periodically to assure adherence to the specified tolerances on dimensions.

(3) *Water absorption apparatus.* A corrosion-resistant, flat-bottomed pan approximately 2 inches in depth, and a cabinet or moist room maintained at a temperature between 65° and 80° F. (18.5° and 26.5° C.), and relative humidity of at least 90 percent.

(4) *Extrusion test assembly.* ASTM Designation D 915 and consisting of the following:

(*a*) *Testing model.* A cylindrical hardened steel mold 4.5 inches long having an outside diameter of 3 inches, and an inside diameter of 2.07±0.01 inch at the large end tapered to 2.020±0.001 inch at a point 2.25 inches from the bottom and continuing at this diameter to the bottom.

(*b*) *Test plunger.* A cylindrical, hardened tool steel plunger having a diameter of 1.125±0.001 inch on the testing end for a distance of 2 inches, and then a diameter of 1.250±0.005 inch for a distance of 4 inches, for an overall length of 6 inches.

(*c*) *Test plunger guide.* A cylindrical steel guide or sleeve having an outside diameter of 2.010±0.005 inch, an inside diameter of 1.265±0.005 inch, and a length of 4 inches.

(*d*) *Testing ring.* A hardened tool steel, annular plate, 0.25 inch in thickness, having a circular opening with a diameter of 1.125±0.001 inch. The cutting edge of the aperture is 0.0625 inch in thickness, and the aperture is tapered outward from the cutting edge at an angle of approximately 30° with the vertical. It is fitted with a "seat" 2.018±0.001 inch in diameter and 0.05 inch in height.

(*e*) *Testing ring support.* A testing ring support of the Hubbard-Field type, or equivalent, adequate to support other parts of the extrusion test assembly.

(5) *Testing machine.* A loading device of at least 10,000-pound capacity, capable of applying a load through a uniform motion of the testing head at a rate of 1 inch per minute. It is fitted with a device for continuously measuring the applied load, and is equipped with a maximum load indicator.

(6) *Drying oven.* A drying oven capable of maintaining a temperature of 140°±5° F. (60°±3° C.)

(7) *Measuring device.* A measuring device that is accurately calibrated and equipped to determine heights and diameters of test specimens to the nearest 0.01 inch.

(8) *Water determination apparatus.* Apparatus conforming to the requirements of the *Standard Method of Testing for Water in Petroleum Products and Other Bituminous Materials* (ASTM Designation: D 95). The test apparatus is described in paragraph 5–14.

(9) *Balance.* A balance having a capacity of 2,000 grams or more and sensitive to 0.5 gram or less or a balance having a capacity of 500 grams or more and sensitive to 0.05 gram or less.

(10) *Sieves.* No. 4 and No. 10 sieves.

(11) *Containers.* Metal containers with tight-fitting lids for storing and "aging" mixtures.

(12) *Mixing pans, thermometers.*

b. Materials.

(1) *Distilled water.* Used in preparing mixtures and in absorption tests.

(2) *Xylol.* Used as the solvent for determining the moisture content of soil and soil-bituminous mixtures in accordance with ASTM Method D 95.

(3) *Bituminous material.* Emulsified asphalt, cutback, or other liquid asphalt, or tar; the type and grade as specified.

c. Preparation of Soil. The air-dry soil is pulverized to separate the soil particles without reducing the particle size, and screened through a No. 4 sieve. The percentage retained on the No. 4 sieve is recorded. The soil passing the No. 4 sieve is screened through the No. 10 sieve. If soil-binder clusters are retained on the No. 10 sieve, they are further pulverized to pass the No. 10 sieve. The material passing the No. 4 sieve and retained on the No. 10 sieve is then combined and intimately mixed with the material passing the No. 10 sieve. The soil thus prepared is stored in tightly closed containers. The water absorption, expansion, and extrusion tests are conducted on the total material passing the No. 4 sieve.

d. Moisture Content and Dry Weight of Soil.

(1) *Moisture content.* The moisture content of the air-dried soil is determined by testing 100 grams of the prepared soil by the procedures outlined in paragraphs 2–53 and 2–54.

(2) *Dry weight.* The weight of dry soil to be used in the water absorption, expansion, and extrusion tests is calculated as follows:

$$\text{Weight of dry soil} = \frac{\gamma_w}{1+\frac{w}{100}}$$

e. Preparation of Mixtures.

(1) *Tar and liquid asphalt.*

(*a*) If the addition of water is required, the air-dried soil described in *c* above is weighed out in 1,000-gram portions (calculated on the dry-soil basis) to supply sufficient mix materials for the required number of test specimens. The weighed quantity of soil is placed in the bowl of the mechanical mixer. The specified amount of distilled water, based upon the calculated dry weight of the soil and with due allowance for the determined moisture present, is added and roughly distributed by cutting and stirring with a stiff knife. The mixer is started and mixing continued for a total mixing period of 5 minutes, stopping when necessary to clean the sides of the mixing bowl and blade. Time spent in scraping the blade and sides of mixing bowl is not included in the mixing time. The soil-water mixture is then placed in a tightly closed metal container and allowed to stand for a period of 16 to 24 hours.

(*b*) The aged soil-water mixture or air-dried soil is placed in 1,000-gram portions (calculated in the dry-soil basis) in the bowl of the mechanical mixer. The specified amount of bituminous material is added into a depression previously made in the prepared soil-water mixture and roughly stirred into the mixture by means of a stiff knife. The bowl is attached to the mixer, the mixer started, and mixing continued for a total mixing period of 5 minutes, stopping when necessary to clean the sides of the mixing bowl and blade. Extra mixing may be performed by hand, if necessary, for an additional 5-minute period, using a rubber tipped pestle. The soil-bituminous mixture is placed in a tightly closed container and allowed to stand for a period of 4 hours. The mixture is formed into specimens the same day the mix is prepared (*f* below).

(2) *Emulsified asphalt.* The air-dried soil prepared as described in *c* above is weighed out in 1,000-gram portions (calculated on the dry-soil basis) to supply sufficient mix material for the required number of test specimens. The weighed quantity of soil is placed in the bowl of the mechanical mixer, the machine started at slow speed, and the specified amount of distilled water added to the moving mass Mixing is continued for a total elapsed period o 5 minutes, stopping when necessary to clean the side of the mixing bowl. Time spent in scraping the side of mixing bowl is not included in the mixing time The specified amount of emulsified asphalt is adde to the wetted soil and stirring continued for a minimum of 5 minutes or a maximum of 10 minutes s as to produce a mixture having visual homogeneity The completed mixture is placed in tared shallo

NOTE

Certain soils have a tendcy toward formation of hard clods during drying. Special precautions are necessary at times to improve the handling characteristics and uniformity of the mixture during compaction. In such cases, as soon as the consistency of the mixture will permit, it is cut with a spatula into dice-like pieces having approximately ½- to ¾-inch edges. These diced cubes are stirred occasionally during drying. When the cubes have dried to just below the plastic limit consistency of the soil, they are placed in the bowl of the mechanical mixer and mixed at the second speed of the machine until a discrete, uniform, granular mass is obtained. This worked mass is placed in tared shallow pans and allowed to continue to air-dry, while stirring with a spatula at intervals, to the specified moisture content.

(3) *Moisture content of mixture.* The moisture content of the aged soil-bituminous mixture is determined by testing 100 grams of the mixture in the water determination apparatus, using xylol as a solvent. The moisture content is calculated as follows:

Moisture content, M, percent by weight

$$=M=\frac{V(100+B)}{(100-V)}$$

Where:

V=milliliters of water collected in the water determination apparatus, and

B=percentage by weight (calculated on the dry soil basis) of bituminous material added in preparing the mixture.

If the percentage of moisture does not vary from the specified moisture content by more than 0.15, the soil-bituminous moisture is formed into specimens as described in *f* below. If the moisture content does not fall within this allowance tolerance, the mixture is discarded and a new mixture prepared.

NOTE

The percentages of moisture and of bituminous material are calculated on the basis of calculated dry soil=100 percent; that is, 7 percent moisture, 5 percent bituminous material means—

Calculated dry soil	100.0 grams
Hygroscopic plus added moisture	7.0 grams
Bituminous material	5.0 grams

f. Preparation of Specimens.

(1) *Number.* Three specimens are prepared for each test result desired, making each test result the average of three readings. Each of the three readings is recorded.

(2) *Molding.* Portions of the soil-bituminous mixture, each sufficient to form a specimen 2.00 inches in diameter by 2.00±0.05 inch in height, are weighed and formed into specimens as follows:

(*a*) The short compaction plunger is inserted approximately 1 inch into the bottom of the compaction mold suitably supported to maintain its position during initial compaction. The weighed mixture is placed in the compaction mold.

(*b*) Initial compaction is obtained by 25 blows with the tamper. The tamper is allowed to fall freely from a height of 6 inches above the bottom of the specimen being formed, and is rotated between application of the blows. The long compaction plunger is then inserted into the top of the compaction mold, the support for the mold removed, and the entire assembly placed in the testing machine.

(*c*) Final compaction is obtained by compressing the specimen at a constant rate of 1 inch per minute until a total load of 6,000 pounds is indicated. This load is maintained for 2 minutes. The specimen is removed from the mold, the top of the specimen as tamped is marked, and the specimen then weighed and measured for average height and diameter. The average height of the specimen should be 2.00±0.5 inch. Any specimens not conforming to the specified tolerance are discarded.

(3) *Density.* The density is expressed on the basis of the calculated weight of dry soil in the specimen, assuming that no change has occurred in the composition of the mixture during compaction:

$$W_d=\frac{100W_s}{100+M+B}$$

$$D=\frac{W_d\times 62.43}{v\times 16.39}=\frac{W_d}{v}\times 3.81$$

Where:

W_d= calculated weight of dry soil in specimen, in grams

W_s=weight of specimen in grams

M=percentage by weight of moisture in mixture molded into specimens (*e*(3) above)

B=percentage by weight (calculated on the dry soil basis) of bituminous material added in preparing the mixture

D=density in pounds of dry soil per cubic foot, and

v=volume of specimen in cubic inches.

g. Curing of Specimens.

(1) *Soil and emulsified asphalt.* Unless otherwise specified, specimens containing emulsified asphalt are tested in the cured state. Prior to testing

for moisture absorption, expansion, and extrusion value, the specimens are cured as follows:

(*a*) the soil-emulsified asphalt specimens are placed in an oven maintained at 140°±5° F. (60°±3° C.) until 80 to 90 percent (unless otherwise specified) of the moisture at the time of molding has been removed. This point is determined on a control specimen chosen from each group of three specimens. At the completion of the curing period, the weight, average height, and the diameter of each specimen are recorded.

(*b*) The control specimen, after removal from the curing oven, is placed in a desiccator for at least 30 minutes (to reach laboratory temperature) and then weighed. All other specimens are removed at the same time and placed in a desiccator arrangement similar to that used for the control specimen, to insure the same degree of drying.

(*c*) The curing of specimens is conducted only during times when the laboratory is in operation. At times when control checks cannot be carried out, all specimens are removed from the oven and placed in a sealed container to retard further loss in weight.

(2) *Soil and asphalt or tar.* Soil-asphalt and soil tar specimens are tested in the uncured state, in the cured state, or both, as specified. If tests on cured specimens are required, they are cured prior to testing for moisture absorption, expansion, and extrusion value, as follows:

(*a*) The specimens are dried to constant weight in an oven maintained at 140°±5° F. (60° ±3° C.). Constant weight for this purpose is reached when the loss in weight during a 6-hour period in the oven is 0.3 gram or less. From each set of three specimens, a control specimen is chosen to determine when constant weight has been reached.

(*b*) All specimens are placed in an oven maintained at 140°±5° F., for an initial curing period of 40 to 42 hours, after which all specimens, removed from the oven, are placed in a desiccator for at least 30 minutes to reach laboratory room temperature. The control specimens are weighed. All specimens are again placed in the oven, and after an interval of 6 hours, removed, and cooled to room temperature in a desiccator, and the control specimens reweighed. Further weight readings, based on 6-hour periods in the oven, are made until the control specimens have been dried to constant weight as defined in (*a*) above. After the initial curing period, when the last weight readings on a given day have been made, all the specimens are removed from the oven and placed in a desiccator until the start of the next working day.

(*c*) When a control specimen has been dried to constant weight, all specimens represented by that control specimen are removed from the desiccator and the weight, average height, and diameter o each specimen recorded.

h. Procedure for Water Absorption and Expansion tests.

(1) *Absorption cabinet.* The water absorptio test is performed in an absorption cabinet or mois room maintained at a temperature between 65° an 80° F. (18° and 27° C.) and a relative humidity o at least 90 percent.

(2) *Exposure and measurement of specimens* The bottom of the specimens, as tamped, is place directly on the bottom of a flat-bottomed pan, an the distilled water in the pan is adjusted to and maintained at a height of 1 inch above the bottom of th specimens. After 7 days, the specimens are remove from the absorption cabinet or moist room and th free surface moisture removed by quick blotting The weights and average diameters of the bottom o the specimens are recorded.

(3) *Calculation of water absorption.* Water absorption (in percent)

$$=\frac{W_2-W_1}{W_d}\times 100$$

Where:

W_1=weight before absorption
W_2=weight after absorption, and
W_d=calculated weight of dry soil specimen.

(4) *Calculation of expansion.* Expansion (i percent)

$$=\frac{D_2^3-D_1^3}{D_1^3}\times 100=\left(\frac{D_2^3}{D_1^3}-1\right)100.$$

Where:

D_1=diameter before absorption, and
D_2=average diameter of bottom of specime after absorption.

i. Procedure for Extrusion Test.

(1) *Specimens.* The extrusion test is performe at room temperature on specimens that have bee subjected to the water absorption test.

(2) *Assembly.* After the specimen has been r moved from the absorption cabinet, weighed, an measured, it is placed in the testing mold with t face downward that was down during the absor tion test. The specimen is inserted into the large en of the testing cylinder, being gradually forced, necessary, by use of the compaction plunger unt seated against the testing ring. The testing plung and guide are inserted into the large end of t testing mold until resting on top of the test specime

(3) *Test.* The entire test assembly is centered o the platform of the testing machine. Load is applie through the test plunger, which is moved at a co stant rate of 1 inch per minute. The extrusion val is the maximum total load required to cause failu of the test specimen.

j. Report. The following data are reported as the average of three determinations:

(1) Water absorption during the absorption test.

(2) Expansion during the absorption test.

(3) Extrusion value after the absorption test.

(4) Additional data as may be specified.

5-14. Method of Testing for Water

This method (ASTM Designation D 95*) is used to determine the water content of bituminous materials by distillation with a water-immiscible, volatile solvent. The method is suitable for a variety of bituminous materials, but it is especially applicable to crude petroleums, tars, and products derived from them, such as fuel oils, road oil, creosotes, road tars, and asphalts. The material is heated under reflux with a water-immiscible solvent which co-distills with the water in the sample.

a. Apparatus.

(1) *General.* The condensed solvent and water are continuously separated in a trap, with the water settling into a graduated section of the trap and the solvent returning to the still. The apparatus consists of a metal still or glass flask, heated by suitable means and provided with a reflux condenser discharging into a trap connected to the still or flask. The trap serves to collect and measure the condensed water and to return the solvent to the still. The type of distilling apparatus used is not an essential feature of this method, but glass is generally used for petroleum products and mètal for road materials and tars.

(2) *Still.* The still specified in the *Standard Method of Testing for Water in Petroleum Products and Other Bituminous Materials*, American Society for Testing and Materials Designation D 95, is a vessel having a nominal capacity of 500 to 1000 ml and a short neck accommodating the reflux tube of the trap.

(3) *Heat source.* The burner used with the metal still is a ring gas burner of 4 inches (100-mm) inside diameter. With the glass flask, an ordinary gas burner or electric heater may be used as the source of heat.

(4) *Condenser.* The condenser is a water-cooled, reflux, glass-tubed type, having a condenser jacket not less than 400-mm (15¾ inches) in length, with an inner tube 9.5 to 12.7 mm (⅜-½ inch) outside diameter. The end of the condenser to be inserted in the trap is ground off at an angle of 30° ± 5° from the vertical axis of the condenser.

(5) *Trap.* The trap is made of well-annealed glass constructed as specified in ASTM D 95, and is graduated in 0.1 ml divisions from 0 to 2 ml and 0.2 ml divisions from 2 to 10 ml. The error of the indicative capacity from 0 to 2 ml must not be greater than plus or minus 0.05 ml and from to 2 to 10 ml must not exceed plus or minus 0.10 ml.

b. Solvent.

(1) *General use.* An aromatic solvent is preferred since it has a high solvency and dispersing power for most bituminous materials. Xylol or a blend of 20-percent benzol and 80-percent xylol is recommended.

(2) *Petroleum derivatives.* The solvent used when testing petroleum products or bituminous materials derived from petroleum is a petroleum derivative, such as petroleum naphtha, and conforms to the following distillation requirements:

5 percent boiling at a temperature not below 194°F. (90°C.) nor above 212°F.(100°C.), and

90 percent distilling below 410°F. (210°C.).

(3) *Tar derivatives.* The solvent used when testing bituminous materials derived from coal tar, water-gas tar, and similar materials, is the aromatic solvent ((1) above).

c. Sample. The sample must be thoroughly representative of the materials to be tested and the portion used for the test thoroughly representative of the sample itself. Deviation from this requirement is not permitted. The sample must be measured to an accuracy of ± 1 percent and added to the still. Ordinary liquid samples are measured in a graduated cylinder. Rinse the material adhering to the cylinder into the still with one 50 ml and two 25 ml portions of solvent. Drain the solvent thoroughly after the transfer and each rinsing. Weigh solid and semisolid materials directly into the still and all 100 ml of solvent.

NOTE

The difficulties in obtaining proper representative samples for this determination are unusually great and the importance of sampling cannot be too strongly emphasized.

d. Procedure.

(1) *Water content of sample.* Base the size of the test portion on an estimated water content of the sample so that the water yield does not exceed the capacity of the trap.

NOTE

In special cases where the water content is great and it is not desirable to reduce the size of the sample, a distilling tube receiver graduated from 0 to 2 ml in 0.1 ml divisions and from 2 to 25 ml in 0.2 ml divisions may be used. (See the Standard Method of Testing Emulsified Asphalts: ASTM Designation D 244).

(2) *Connections.* The connections between the still or flask, trap, and condenser must be made by means of tight-fitting stoppers or ground glass joints. The end of the condenser inserted in the trap is ad-

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submerged to a depth of not more than 1-mm below the surface of the liquid in the trap after distillation conditions have been established. When the metal still is used, a heavy paper gasket moistened with the solvent is inserted between the lid and flange before attaching the clamp. A loose cotton plug is inserted in the top of the condenser tube to prevent condensation of atmospheric moisture in the condenser tube.

(3) *Application of heat.* Heat is applied and so regulated that the condensed distillate falls from the end of the condenser at the rate of 2 to 5 drops per second. The ring burner used with the metal still should be placed about 3 inches above the bottom of the still at the beginning of the distillation and gradually lowered as the distillation proceeds.

(4) *Duration of test.* The distillation is continued at the specified rate until no water is visible on any part of the apparatus except at the bottom of the trap. This operation usually requires less than an hour. A persisting ring of condensed water in the condenser tube is removed by increasing the rate of distillation for a few minutes. The trap and contents are allowed to cool to room temperature. Dislodge any water adhering to the trap sides and make sure it falls into the trapped water. Read the volume of water to the nearest smallest division.

e. Calculations. The volume of condensed water measured in the trap at room temperature multiplied by 100 and divided by the volume of the sample used is the percentage of water, or

$$\text{Water (percent)} = \frac{\text{Volume of water in trap}}{\text{Volume (or weight) of sample}} \times 1$$

Note that water-soluble material may also be mea ured as water.

f. Precision. Two tests of the sample materi should agree within the following limits:

Volume of water collected	*Repeatability (same operator and apparatus)*	*Reproducibility (Differe operator and apparatus*
0 to 1 ml	0.1 ml	0.2 ml
1 to 25 ml	0.1 ml or ± 1% of the mean.	0.2 ml or ± 5% of the mean.

5–15. Acceptability Requirements

For purposes of designing soil-bitumen mixtures, t following test values are suggested:

a. Extrusion value before the absorption test, 1,0 pounds minimum.

b. Extrusion value after the absorption test, 4 pounds minimum.

c. Expansion during the absorption test, 5 perce maximum.

d. If it is also desired to include the percent wat absorption during the absorption test, a maximu of 7 percent is suggested. The above values are bas on the recommended procedure of testing uncur specimens. No recommended values have been esta lished for results on cured specimens. In cases doubt it should be borne in mind that a sligh leaner rather than slightly richer soil-asphalt mi ture is preferable.

APPENDIX A

REFERENCES

A-1. Military Publications

a. Army Regulations.

AR 310-25 Dictionary of United States Army Terms

AR 310-50 Authorized Abbreviation and Brevity Codes

AR 415-30 Troop Construction for the Air Force. (AFR 88-12)

b. Field Manuals.

FM 5-34 Engineer Field Data

FM 5-35 Engineers' Reference and Logistical Data

c. Technical Manuals.

TM 5-330/AFM 86-3, VOL II Planning and Design of Roads, Airbases, and Heliports in the Theater of Operations

TM 5-331A Utilization of Engineer Construction Equipment; Volume A—Earthmoving, Compaction, Grading, and Ditching Equipment

TM 5-331D Utilization of Engineer Construction Equipment; Volume D—Asphalt and Concrete Equipment

TM 5-337 Paving and Surfacing Operations

TM 5-349 Arctic Construction

TM 5-545 Geology

TM 5-624 Roads, Runways and Miscellaneous Pavements; Repairs and Utilities:

d. DA Pamphlet 525-5 Dust Control

e. DA Technical Bulletin 5-337-1 Central Mix Plant and Slip-Form Paver

A-2. Other Technical Publications

American Association of State Highway Officials. *Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part II.* Washington DC, 1966

American Society for Testing and Materials, 1969 Book of ASTM Standards, Part 10—*Concrete and Mineral Aggregates*, and Part 11—*Bituminous Materials for Highway Construction, Waterproofing, and Roofing; Soils; Skid Resistance.* Philadelphia, Pennsylvania, 1969

The Asphalt Institute, Manual Series No. 4 (MS-4): *The Asphalt Handbook,** College Park, Maryland, 1965

The Asphalt Institute, Manual Series No. 10 (MS-10): *SOILS MANUAL for Design of Asphalt Pavement Structures**

The Asphalt Institute, Manual Series No. 13 (MS-13): *Asphalt Surface Treatments and Asphalt Penetration Macadam*

The Asphalt Institute, SS-1; *Specifications and Construction Methods for Hot Mix Asphalt Paving**

*Concrete Inspection Manual** by Kelley, Berlin, and Clair

*Data Book for Civil Engineers, Volume II, Specifications and Costs** by Seelye

*Handbook of Heavy Construction** by Stubbs

*Highway Engineering** by Ritter and Paquette

Portland Cement Association booklet, *Design and Control of Concrete Mixtures*, Eleventh Edition

*Reinforced Concrete Design Handbook** The American Concrete Institute

*Soil Mechanics, Foundations and Earth Structures** by Tschebotarioff

*Engineering Materials*** by The Committee on Engineering Materials

*CRSI Design Handbook*** by Concrete Reinforcing Steel Institute

Properties of Concrete by A. M. Neville, John Wiley & Sons, Inc.

Concrete by G. E. Troxell and H. E. Davis, McGraw-Hill Book Company

Concrete Manual, United States Department of the Interior, 1963

Concrete Construction Handbook by J. J. Waddell, McGraw-Hill Book Company, 1968

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*Contained in BOOK SET: Engineer Construction Group FSN 7610-664-0437.

**Contained in BOOK SET: Field Army, Engineer Section FSN 7610-664-0438.

APPENDIX B

UNIFIED SOIL CLASSIFICATION SYSTEM

The Unified Soil Classification System is a method of describing the different soils based on the soils' physical characteristics. The system was adopted by the U.S. Army Corps of Engineers (and the Department of Interior Bureau of Reclamation) in January 1952. The three tables, B-1, B-2, and B-3, are included to aid in identifying the soils from the different test results, and to illustrate the engineering characteristics pertinent to roads and airfields, and embankments and foundations. The tables list some of the limiting values associated with the tests described in this manual. The values should guide the technician in his testing. A great discrepancy between a test result and the values given in the tables for that particular soil type should alert the technician to the possibility of some inaccuracy or error either in the testing or in the identification of the soil. Figure B-1 illustrates the process of identifying soils through the tests. A more detailed discussion of the system is given in TM 5-330.

Table B-1. Unified Soil Classification System.

Table B-2. Characteristics Pertinent to Roads and Airfields.

Table B-3. Characteristics Pertinent to Embankments and Foundations.

Figure B-1. Identification of soils under soil classification system.

[Located in back of manual]

APPENDIX C

USEFUL INFORMATION

1 inch=2.54 centimeters=25.4 millimeters
12 inches=1 foot=0.3048 meter
1 square foot=144 square inches
1 square yard=9 square feet=1,296 square inches
1 acre=43,560 square feet
1 acre, if square=approximately 208.7 feet on each side
1 cubic foot=1,728 cubic inches=7.48 United States gallons
1 cubic yard=27 cubic feet=0.7646 cubic meter
1 cubic meter=1.308 cubic yards=35.31 cubic feet=1,000 liters
1 United States gallon=231 cubic inches=0.1337 cubic foot=4 quarts=8 pints
1 United States gallon of water weighs 8.33 pounds
1 British Imperial gallon=1.2 United States gallons
1 gross or long ton=2240 pounds=1.12 net tons
1 net or short ton=2000 pounds
1 pound=16 ounces=256 drams=7000 grains=453.6 grams
1 kilogram=1000 grams
1 atmosphere=14.7 pounds per square inch=30 inches Mercury=33.95 feet of water at 62° F.
1 pound per square inch=144 pounds per square foot=0.068 atmosphere=2.042 inches mercury=2.1 feet of water at 62° F.
1 cubic foot of water at 62° F. weighs 62.36 pounds
Water freezes at 32° F. or at 0° C.
Water boils at 212° F. or 100° C. (sea level and 14.7 pounds per square inch pressure).

$$°\text{F.}=\frac{9}{5}(°\text{C.})+32$$

Circumference of a circle = diameter × 3.14
Area of a circle = diameter squared by 0.7854
1 foot-pound = unit of work or the work required to lift 1 pound through a vertical distance of 1 foot.

*Relation between A.P.I. degrees and specific gravity (liquids—lighter than water)**

A.P.I. Gravity Degrees	*Specific Gravity, 60°/60°*
10	1.000
15	0.9655
20	0.9333
25	0.9032
30	0.8750
35	0.8485
40	0.8235
45	0.8000

*A.P.I. Gravity Degrees $=\frac{141.5}{\text{sp gr } 60°/60°}-131.5$

Multiply	*By*	*To obtain*
Atmospheres	14.70	Pounds per square inch
Centimeters	0.3937	Inches
Cubic feet	7.481	United States gallons
Cubic feet	1728	Cubic inches
Cubic meters	35.51	Cubic feet
Cubic yards	27	Cubic feet
Feet	0.3048	Meters
Feet	30.48	Centimeters
Gallons	231	Cubic inches
Gallons	0.134	Cubic feet
Inches	2.54	Centimeters
Kilograms	1000	Grams
Kilograms	2.205	Pounds
Liters	0.03531	Cubic feet
Liters	0.2642	Gallons
Liters	1000	Cubic centimeters
Miles	5280	Feet
Pounds	453.6	Grams

APPENDIX D

METRIC CONVERSION TABLE

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1 centimeter = 0.03281 feet
1 centimeter = 0.3937 inches
1 cubic centimeter = 0.06102 cubic inches
1 inch = 2.540 centimeters
1 cubic inch = 16.39 cubic centimeters
1 gallon = 3785 cubic centimeters
1 gallon = 3.785 liters
1 liter = 61.02 cubic inches
1 liter = 0.2642 gallons
1 kilogram = 2.205 pounds
1 pound = 453.6 grams

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			GC	Clayey
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			SC	Clayey
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			CL	Inorganic gravell lean cl
			OL	Organic plastic
	Silts and Clays Liquid limit is greater than 50		MH	Inorganic fine sa
			CH	Inorganic
			OH	Organic organic

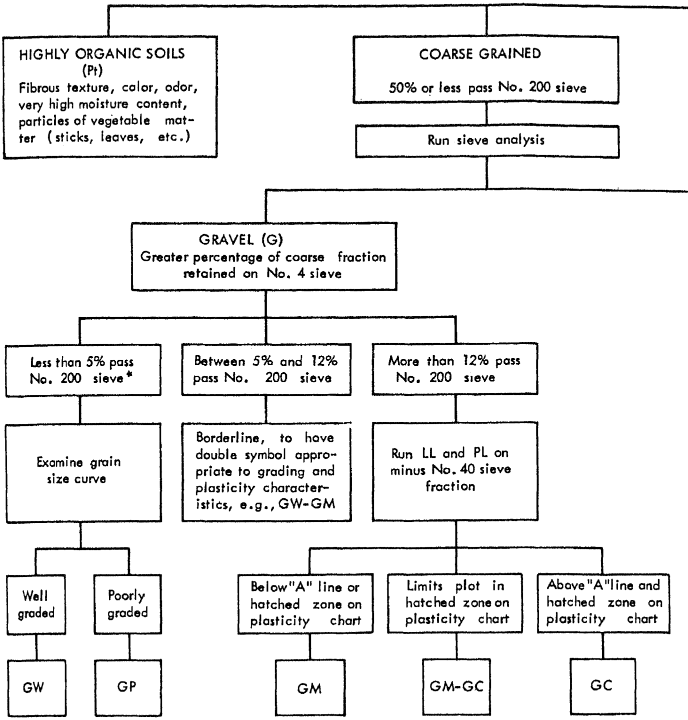
Major Divisions (1)	(2)	Letter (3)		Symbol Hatching (4)	Symbol Color (5)	Name (6)	Value as Foundation When Not Subject to Frost Action (7)	Value as Base Directly under Bituminous Pavement (8)	Potential Frost Action (9)	Compre Exp
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		⁘	Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Good	None to very slight	Almost
		GP		⁘		Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Poor to fair	None to very slight	Almost
		GM	d	▮	Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Fair to good	Slight to medium	Very sl
			u	▮			Good	Poor	Slight to medium	Slight
		GC		▨		Clayey gravels, gravel-sand-clay mixtures	Good	Poor	Slight to medium	Slight
	SAND AND SANDY SOILS	SW		⁘	Red	Well-graded sands or gravelly sands, little or no fines	Good	Poor	None to very slight	Almost
		SP		⁘		Poorly graded sands or gravelly sands, little or no fines	Fair to good	Poor to not suitable	None to very slight	Almost
		SM	d	⁘	Yellow	Silty sands, sand-silt mixtures	Good	Poor	Slight to high	Very sl
			u	⁘			Fair to good	Not suitable	Slight to high	Slight medium
		SC		▨		Clayey sands, sand-clay mixtures	Fair to good	Not suitable	Slight to high	Slight
FINE GRAINED SOILS	SILTS AND CLAYS LL < 50	ML		▥	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Fair to poor	Not suitable	Medium to very high	Slight
		CL		▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Fair to poor	Not suitable	Medium to high	Medium
		OL		▥		Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Medium to high	Medium
	SILTS AND CLAYS LL > 50	MH		▥	Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Medium to very high	High
		CH		▨		Inorganic clays of high plasticity, fat clays	Poor to very poor	Not suitable	Medium	High
		OH		▨		Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Medium	High
HIGHLY ORGANIC SOILS		Pt		▤	Orange	Peat and other highly organic soils	Not suitable	Not suitable	Slight	Very hi

Notes:

1. Column 3, Division of GM,and SM groups into subdivisions of d and u are for roads and airfields only; subdivision is on basis of Atterberg limi
plasticity index is 6 or less; the suffix u will be used when the liquid limit is greater than 28.
2. Column 7, values are for subgrades and base courses except for base course directly under bituminous pavement.
3. In column 8, the term "excellent" has been reserved for base materials consisting of high quality processed crushed stone.
4. In column 9, these soils are susceptible to frost as indicated under conditions favorable to frost action described in the text.
5. In column 12, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and th
are listed, because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of
   a. Processed base materials and other angular materials. Steel-wheeled rollers are recommended for hard angular materials with limited fines or
   to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor -- total weight in excess of 30,000 lb.
      Rubber-tired equipment -- wheel load in excess of 15,000 lb, wheel loads as high as 40,000 lb may be necessary to obtain the required densi
      Sheepsfoot roller -- unit pressure (on 6- to 12-sq-in. foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necess
      should be at least 5 per cent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
6. Column 13, unit dry weights are for compacted soil at optimum moisture content for modified AASHO compactive effort.

Major Divisions (1)	(2)	Letter (3)	Symbol		Name (6)	Value for Embankments (7)	Permeabilit Cm Per Sec (8)
			Hatching (4)	Color (5)			
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	░	Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GP	░		Poorly-graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GM	▥	Yellow	Silty gravels, gravel-sand-silt mixtures	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-6}
		GC	▨		Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}
	SAND AND SANDY SOILS	SW	░	Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$
		SP	░		Poorly-graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$
		SM	▥	Yellow	Silty sands, sand-silt mixtures	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-6}
		SC	▨		Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core for flood control structures	$k = 10^{-6}$ to 10^{-8}
FINE GRAINED SOILS	SILTS AND CLAYS LL < 50	ML	▥	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-6}
		CL	▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}
		OL	▥		Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}
	SILTS AND CLAYS LL > 50	MH	▥	Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor stability, core of hydraulic fill dam, not desirable in rolled fill construction	$k = 10^{-4}$ to 10^{-6}
		CH	▨		Inorganic clays of high plasticity, fat clays	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-6}$ to 10^{-8}
		OH	▨		Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}
HIGHLY ORGANIC SOILS		Pt	▤	Orange	Peat and other highly organic soils	Not used for construction	

Notes: 1. Values in columns 7 and 11 are for guidance only. Design should be based on test results.
2. In column 9, the equipment listed will usually produce the desired densities with a reasonable number of passes when moistu
3. Column 10, unit dry weights are for compacted soil at optimum moisture content for Standard AASHO (Standard Proctor) compact


HIGHLY ORGANIC SOILS (Pt)
Fibrous texture, color, odor, very high moisture content, particles of vegetable matter (sticks, leaves, etc.)
COARSE GRAINED
50% or less pass No. 200 sieve
Run sieve analysis
GRAVEL (G)
Greater percentage of coarse fraction retained on No. 4 sieve
Less than 5% pass No. 200 sieve*
Between 5% and 12% pass No. 200 sieve
More than 12% pass No. 200 sieve
Examine grain size curve
Borderline, to have double symbol appropriate to grading and plasticity characteristics, e.g., GW-GM
Run LL and PL on minus No. 40 sieve fraction
Well graded
Poorly graded
Below "A" line or hatched zone on plasticity chart
Limits plot in hatched zone on plasticity chart
Above "A" line and hatched zone on plasticity chart
GW
GP
GM
GM-GC
GC

Note: Sieve sizes are U.S. Standard.

* If fines interfere with free draining properties use double symbol such as GW-GM, etc.